

Research Report
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RUTTING: A CASE STUDY
(US 23; 1.5 Miles North of Louisa)

by

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INTRODUCTION

This case study of rutting in an asphaltic concrete pavement makes recourse to trenching full width and full depth to expose and observe the entire cross section of the structure. Rutting occurred in the vicinity of Station 59+00 on US 23, 1.5 miles north of Louisa, during August and September 1983 when traffic was diverted onto newly constructed northbound lanes (without final surface and shouldering) while excavation into the hillside was completed for the southbound lanes. Earth movers crossed the northbound lanes going to and from a waste area. Coal trucks (Figures 1 and 2) slowed, stopped, and crept where rutting was greatest. Some rutting had occurred throughout.

Station 59+00 was chosen because the rutting there exceeded 1.5 inches (Figure 3). Station 56+40 was chosen because rutting was minimal there. The exposures were made September 15, 1983.

Road Rater measurements were made before trenching. The surface profile was recorded as measured deviations from a taut string line.

Two cuts were made about two feet apart and about two feet deep. The material between the cuts was removed with a backhoe. The cuts were made with a Ditch Witch Earth Saw (Figure 4). Each cut was about 4 inches wide. Cutting progressed rapidly and without difficulty.

The first exposure of this type in Kentucky was in 1957 (1). The pavement was on New Circle Road between Winchester Road and Bryan Station Road. The rutting extended through the waterbound macadam base and was largely credited to deformation of the roadbed soil. Although rutting was observed at other notable places in the interim (for example, I 75 north of Rockcastle River (2)), it was assumed that rutting occurred predominately in the soil underneath. That was so until full-depth asphaltic concrete pavements became a design option. In 1978, an upgrade section of the Daniel Boone Parkway (from the Hyden Spur westward) consisting of 18 inches of asphaltic concrete rutted extensively. Trenching disclosed rutting to be mostly in the upper 5 inches of the asphaltic concrete (3). Heavy coal trucks travelled upgrade at slow speeds. I 64, just east of the US-60 exit to Ashland was trenched and examined in 1978 (4). There the situation was similar to New Circle Road except dense-graded aggregate supplanted the waterbound base. At the same time, a rutted hill section of full-depth asphaltic concrete on US 60, at Tip Top nearer to Ashland, was examined (4). There the situation was similar to the Daniel Boone Parkway. Another exposure was made on US 62, south of Cynthiana. There, the layers could not be differentiated adequately. The cuts were made with a jackhammer.

A prerequisite to the design of pavements for KY 80 (Hazard-Watergap) (5) was to minimize rutting under intense coal-haul traffic. To do that, it was necessary to know where rutting occurred and to strengthen the layers of weakness. KY 627 near Boonesboro was instrumented (6) to measure the progress of rutting under trucks hauling limestone quarried at Boonesboro toward Winchester. Coal was hauled toward Boonesboro, but only the Winchester-bound lane was instrumented. Mathematical models have been derived (7, 8), and it is possible to estimate the magnitude of rutting from material tests and repetitions of constituted traffic. Some background documentation mentioned is provided in the appendices for reference.



Figure 1. US 23, South of Ashland Refinery; Coal Trucks (5-axle semi's) Awaiting Entry into Unloading Yard. About 3:00 pm, 37 Trucks, Extending Back beyond the Photo, Were Waiting.



Figure 2. Station 59+00; Site of Greatest Rutting; Chosen for Trenching; Road Rater Tests in Progress. Note Coal Truck at Left and Absence of Shoulder; Dark Spots are Inside Sand Drains.



Figure 3. Profiling Surface in Outer Lane; Station 59+00. Note
Absence of Outside Shoulder Paving.



Figure 4. Ditch Witch Earth Saw (ES 24); Cut Approximately Two Feet deep; 10 to 15 Minutes per Cut.

Analysis of the US-23 rutting was simply a cause-and-effect approach. Plastic soil was found under the dense-graded aggregate where rutting was greatest (Figures 5, 6, 7, and 8). The material there should have been rock subgrade. At Station 56+40, chunky shale (Figure 9) was found under the dense-graded aggregate. The shale was friable, crumbly, and dry. There the rutting was much less, but the subgrade presumably would not qualify for CBR 11, which was the design value.

The pavement was first designed in 1977. It was re-designed in 1982. Presumably, the latter also predated the Slake Durability Index method of rating shales. Mathis (memo to Division of Design, March 3, 1978) recommended a CBR of 11. That value customarily had been reserved for good-quality rock materials; CBR 9 had been assigned to lesser-quality material (containing some shale). A mismatch is evident inasmuch as plastic material was uncovered at the first site and inasmuch as chunky shale was found at the second site.

TRAFFIC

Traffic was diverted onto the northbound lanes on June 7, 1983, and was diverted off on August 4, 1983. This was a period of 58 days.

In May 1983, 1,276 trucks were counted in 16 hours. That expands to 1,914 per day and(or) 857 trucks per day in each direction. Most trucks were coal haulers and were mostly 5-axle semi-trailers hauling toward Cattlettsburg. Relative damage factors for those trucks has been deduced as follows (9, 10):

Five-Axle Semi-trailer:

Gross Weight = 116 kips
Empty Weight = 28 kips
Payload = 88 kips
Front axle = 16 kips
1st Tandem = 50 kips
2nd Tandem = 50 kips
Damage Factor = $2 + 4.5 + 4.5 = 11.0$

Six-Axle Semi-trailer:

Gross Weight = 124 kips
Empty Weight = 30 kips
Payload = 94 kips
Front Axle = 14 kips
Tandem = 44 kips
Tri-Axle = 66 kips
Damage Factor = $1.2 + 2.4 + 2.2 = 5.8$

Assuming 95 percent of the trucks to be five-axle units and 5 percent to be six-axle units, the weighted damage factor would be 10.7 per truck. The EAL's accumulated per day would be $857 \times 10.7 = 9,169.9$. Expanding this through 20 years of 300 days per year of hauling yields 55×10^6 EAL's. To accumulate the design EAL of 7.47×10^6 (239×10^6 EWL's/32) would require $7.47 \times 10^6 / 9,169.9 = 814$ days, or 2.72 years having 300 work days per year.



Figure 5. Station 59+00; Rutting and Thinning in Wheelpaths; Heaving (Upthrusting) between Wheelpaths; Plastic Soil in Subgrade in Foreground; Sandy Material in Background (See Figure 8).



Figure 6. Station 59+00; Inner Lane Unrutted; Sandy Roadbed.



Figure 7. Station 59+00, Brownish Material under Dense-graded Aggregate Base in Inside Lane. Sample for Laboratory Testing Taken from Foreground Contained Appreciable Sand, Sandstone Fragments, and Shale.



Figure 8. Transition from Brownish Soil (leftward) to Blue-Gray Shale (rightward). Sample was More Representative of Sandstone-Derived Material with Some Shale Than of Purer Shaly Material (rightward) under the Outer Lane.



Figure 9. Station 56+40; Lesser Rutting; Chunky, Friable Dark Shale in Roadbed; Relatively Dry; No Free Water.

The 1977⁶ forecast of traffic for this project was 239,677,756 EWL's (7.47×10^6 EAL's). That was not updated for 1983 construction. The estimate did not include axles (all treated as singles) weighing more than 12 kips (24-kip tandem). For this study, a current forecast was made and yielded 288,530,467 EWL's but did not include axles weighing more than 12 kips. At the request of the Division of Design, another estimate (October 11, 1983) including coal trucks but not axles in excess of 12 kips yielded 863,091,642 EWL's (27×10^6 EAL's). Those calculations are included in the Appendix A.

The process of averaging heavily loaded axles with returning unloaded axles biases the EAL and EWL's downward severely. Customarily, unloaded trucks are not weighed and therefore do not bias averages given in W-4 tables. On the other hand, even averaging lightly loaded axles with very heavy axles could diminish the EWL or EAL by one-half or more. Compare here the 55×10^6 EAL's with 27×10^6 EAL's estimated by others by averaging axleloads. It would be more realistic to average damage factors than to average loads.

It may be of interest, comparatively, to know that Section 7 of KY 80 (Hazard - Watergap) (5) west of Martin was designed for 202×10^6 EAL's. Sections 1, 4, 5, and 8 were designed for 108×10^6 EAL's. A section of the Daniel Boone Parkway was designed initially for an estimated $40+ \times 10^6$ EAL's, but that later was revised downward. It appears that realistic estimates including coal-haul traffic have been moderated by some constraints. On the other hand, the most significant moderating factor (yet respecting continued if not increased hauling of coal) is the trend there away from the so-called "Breathitt County Special" (a 3- or 4-axle single unit with sideboards and over-cab cantilever of bed) to 5-axle and 6-axle semi-trailers.

TRANSVERSE PROFILE AND ANALYSIS OF RUTTING

Deviations from string lines are shown in Figures 10 through 13. The maximum depth of rutting was 0.63 inches in the outer wheel path at Station 59+00. There was an apparent heave of 0.75 inches between the two wheelpaths of the outer lane at that point. Thus, the total deformation there was 1.38 inches. Blue-gray shale (plastic) was found at that point.

A 0.25-inch crown (Figure 11) would almost balance the area (volume) of rut with the area (volume) of material heaved.

A rutting analysis was made of the pavement at Station 59+00 using the method reported by Allen (7, 8). Input parameters to the analysis were as follows:

- Subgrade CBR = 4
- Asphaltic Concrete Thickness = 5 inches
- Dense-Graded Aggregate Thickness = 12 inches
- Traffic (one direction)
 - Vehicles -- 3,800/day
 - Trucks -- 27 percent
 - Tire Pressure (cars) -- 30 psi
 - Wheel Load (cars) -- 0.80 kips
 - Tire Pressure (trucks) -- 120 psi
 - Wheel Load (trucks) -- 7.5 kips

Number of Traffic Days = 60.

Figures 10 and 11 show the profile of the asphalt concrete surface. The loaded coal trucks were using the outside lane. The difference between the elevation of the inside wheel track and the peak between the wheel tracks was 1.15 inches. The difference between the elevation of the outside wheel track and the peak was 1.45 inches. For the purpose of analysis, these two were averaged to give a difference of 1.30 inches. Transverse pavement slope or crown was not considered when taking this average. From the rutting program (PAVRUT), the predicted rut depth was 1.26 inches. This is a difference of only 3.1 percent from observed rutting.

If it is assumed that 0.5 inch of rutting would be the maximum allowed before resurfacing, then the rutting life of the pavement was "used up" in approximately 14 days. If 0.75 inch is the assumed maximum allowable rutting, the rutting life was approximately 22 days. In terms of EAL's travelling at 50 miles per hour, the pavement experienced approximately 9.2 million EAL's in 60 days. (It must be remembered that damage factors for rutting are different than for fatigue. Therefore, the EAL's for fatigue will be different.)

It is assumed the present structure is to receive a 3-inch overlay, making a total of 8 inches of asphaltic concrete on 12 inches of dense-graded aggregate. The original structure was designed for 7.47 million EAL's. However, the original structure has already received 9.2 million EAL's. Therefore, an additional 7.47 million EAL's will produce another 0.41 inch of rutting in the original structure. This same design EAL will produce approximately 0.25 inch of rutting in the 3-inch overlay. Adding these two components of rutting yields approximately 0.67 inch of rutting in the new structure.

Rutting at Station 56+40 (Figures 12 and 13) was not analyzed in this way.

ROAD RATER TESTS AND ANALYSES

Deflection measurements were obtained in the outside lane at Site 1 at Station 59+00. Deflection measurements were obtained for a grid consisting of 3-foot transverse spacings and 10-foot longitudinal spacings. Testing for Site 1 was conducted between Stations 58+70 and 59+30. Testing for Site 2 was conducted between Stations 56+20 and 56+40 and covered both lanes.

Pavement behavior may be expressed in two separate ways: 1) as some thickness of a reference modulus of elasticity for asphaltic concrete on the constructed thickness of crushed stone at the estimated subgrade modulus, or 2) as estimated layer moduli for the constructed layer thicknesses.

ESTIMATION OF SUBGRADE MODULUS OF ELASTICITY

Elastic theory and measured deflection measurements were used to estimate the in-place subgrade modulus of elasticity (11, 12, 13). A small degree of variability was observed. A summary of subgrade moduli and CBR's estimated from deflection data are presented in Table 1. Estimated subgrade moduli were converted to CBR by dividing the

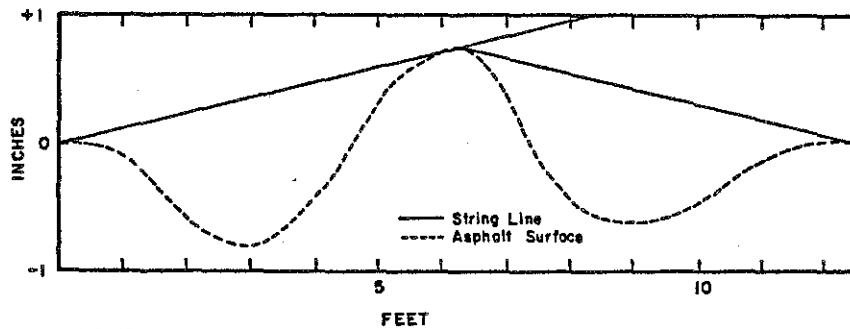


Figure 10. Rutting as Deviations from String Line; Station 59+00; Northbound Lane Only.

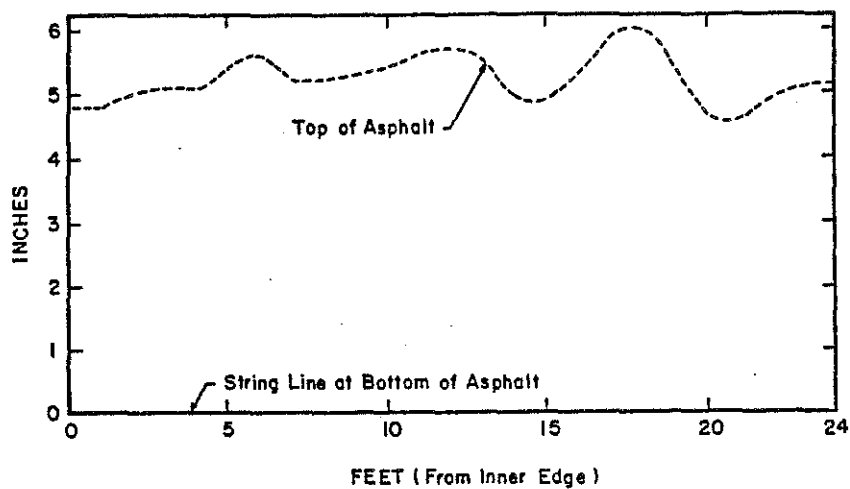


Figure 11. Surface Profile Shown Graphically as Deviations from a String Line at the Bottom of the Asphaltic Concrete; Station 59+00; Both Lanes.

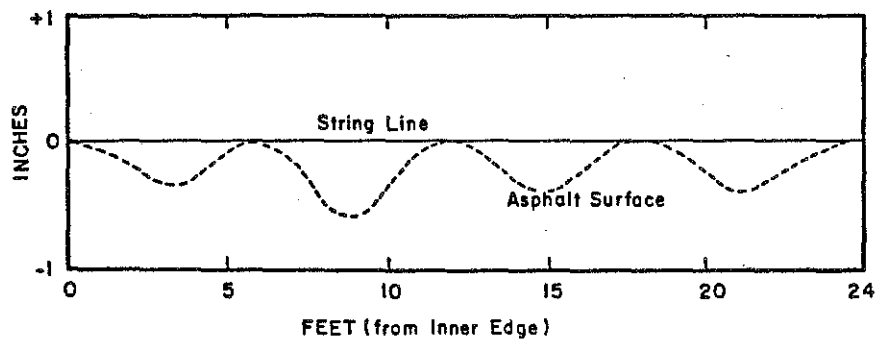


Figure 12. Surface Profile Shown Graphically as Deviations from String Line; Station 56+40; Both Lanes.

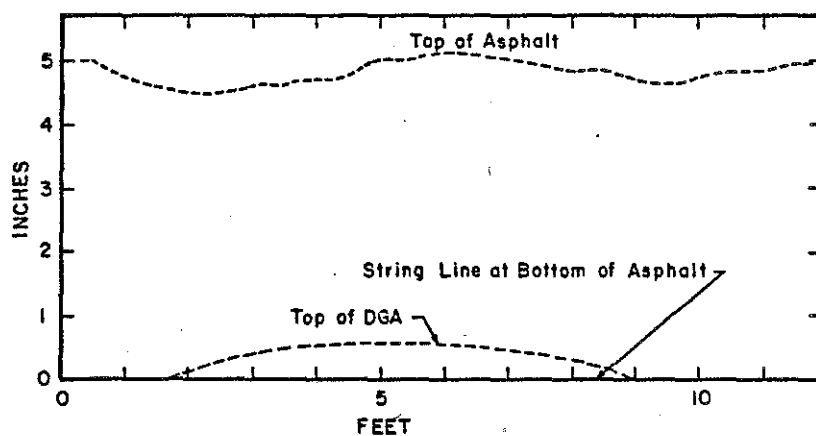


Figure 13. Surface Profile Shown as Deviations from a String Line at the Bottom of the Asphaltic Concrete; Station 56+40; Note Uneven Boundary.

estimated moduli in psi by 1500. The expected spring condition was estimated by multiplying by 0.60. Research has indicated that subgrade moduli estimated during the spring may only be 60 percent of estimated moduli for fall conditions (13, 14).

A review of Table 1 indicated a somewhat weaker subgrade modulus at Site 1 (Station 59+00) than at Site 2 (Station 56+40). Visual observation of field conditions supported those results. Also, laboratory analyses of subgrade samples obtained from Site 1 indicated a soaked CBR of 4, which corresponded well with the estimated "Spring CBR" for Site 1 (Station 59+00).

ESTIMATION OF PAVEMENT MODULUS OF ELASTICITY

Elastic theory was used in combination with the measured first sensor deflections and estimated subgrade moduli of elasticity to estimate the modulus of elasticity for the asphaltic concrete pavement. A summary of estimated moduli of elasticity for the asphaltic concrete pavement is presented in Table 2.

The mean pavement temperature was estimated using the measured pavement surface temperature, time of day, and 5-day average air temperature using procedures developed by Southgate and Deen (13). The mean pavement temperature was then used to estimate the anticipated modulus of elasticity normally associated with asphaltic concrete base pavements (9, 16).

The Kentucky flexible pavement design curves were developed on the basis of an asphaltic concrete modulus of elasticity of 480,000 psi (9, 10). Asphaltic concrete modulus of elasticity is a function of pavement temperature and frequency of loading. At 70 degrees F mean pavement temperature, modulus of elasticity varies from 480,000 psi at 0.5 Hz (near static conditions) to 1,200,000 psi at 25 Hz (normal operating frequency for Road Rater deflection testing). All data presented in Table 2 correspond to moduli at 25 Hz frequency of loading.

Inspection of Table 2 indicates that, on the average, the estimated modulus of elasticity for the asphaltic concrete pavement was approximately 80 percent of the modulus normally anticipated for the measured surface temperature and associated, mean pavement temperature. From a more conservative perspective, the estimated modulus of elasticity is only 41 percent of the normally anticipated modulus of elasticity. Thus, deflection analyses indicate that there has been some apparent reduction or loss in modulus of elasticity relative to that normally anticipated for new quality asphaltic concrete pavements. This is based on the assumption that the pavement had cured and set and that there was no persisting tenderness (17) in the pavement layers.

OVERLAY THICKNESS

For the purpose of overlay design, pavement behavior is expressed as some thickness of reference-quality asphaltic concrete (modulus of 480,000 psi at 0.5 Hz = 1,200,000 psi at 25 Hz at 70 degrees F) on the constructed crushed-stone thickness on the predicted subgrade modulus of elasticity. Each test location was evaluated separately and then combined to estimate an overall required overlay thickness for a design EWL of 2.39×10^8 (EAL = 7.47×10^6).

Calculation of overlay designs was on an individual test point-by-point analysis basis. Each test location was used to predict an in-place

TABLE 1. ESTIMATED SUBGRADE MODULI OF ELASTICITY

	SITE 1 STATION 59+00	SITE 2 STATION 56+40	AVERAGE
Mean Subgrade Modulus (psi)	11,471	18,704	15,087
Mean CBR	7.7	12.5	10.1
Standard Error (psi)	2,034	4,240	
80th-Percentile Subgrade Modulus (psi)*	9,768	15,136	12,452
80th-Percentile CBR	6.5	10.1	8.3
Spring Subgrade Modulus	5,861	9,082	7,472
Spring CBR	3.9	6.1	4.6

*Mean less 0.8416 x standard error

TABLE 2. ESTIMATED PAVEMENT MODULI OF ELASTICITY FROM ROAD
RATER DEFLECTION MEASUREMENTS AT 25 Hz

	SITE 1 STATION 59+00	SITE 2 STATION 56+40
Measured Surface Temperature (degrees F)	78	93.5
Mean Pavement Temperature (degrees F)	76	89.3
Anticipated Modulus of Elasticity (psi) Associated With Mean Pavement Temperature and 25 Hz	900,000	530,000
Mean Pavement Modulus of Elasticity from Deflections (psi)	735,581	416,050
Standard Error (psi)	389,759	263,920
Percent of Anticipated Pavement Modulus of Elasticity	81.7	78.5
80th-Percentile Pavement Modulus of Elasticity (psi)	407,560	193,935
Percent of Anticipated Pavement Modulus of Elasticity	45.3	36.6

Note: 480,000 psi at 0.5 Hz is approximately equal to
1,200,000 psi at 25 Hz at 70 degrees F Mean Pavement
Temperature

behavioral thickness of reference-quality asphaltic concrete and an estimated subgrade modulus adjusted to spring conditions (12). Overlay thicknesses were estimated for each test measurement. Individual overlay thicknesses were then combined and summarized for each test site (Site 1 at Station 59+00 and Site 2 at Station 56+ 40). Overall overlay recommendations were then computed. A summary of overlay thickness requirements is presented in Table 3.

Estimated overlay thickness requirements may seem excessive; however, it should be noted that only 1.28 to 1.70 inches of asphaltic concrete overlay may be attributed to the condition of the asphaltic concrete. Thus, the majority of the recommended overlay thickness is the result of the apparently weak subgrade foundation. The pavement was designed on the basis of an anticipated CBR of 11. Laboratory analyses indicated an existing soaked CBR of 4 for one location. Road Rater deflection analyses indicated (from Table 1) a mean spring CBR of 5 for both test sites. Thus, a very high percentage of the recommended overlay may be directly attributed to the existence of inferior quality subgrade material at those test locations.

It should be noted that overlay estimates are based on the assumption that observed subgrade conditions are representative of the entire pavement section. This assumption may not be completely valid. It is highly possible that those locations may, in fact, be localized conditions as evidenced by excessive rutting measured at those locations but not observed to the same degree for other locations. Therefore, additional deflection testing is recommended to more adequately define the extent of poor foundation conditions.

It should be noted that the overlay designs presented in Table 3 are sufficient only to accommodate a design EAL of 7.47×10^6 . In reality, the accumulation of 7.47×10^6 equivalent 18-kip axleloads may occur in the very near future. Recent estimates using revised traffic volumes have indicated a required design EAL of 9.02×10^6 (EWL = 288,530,467). More realistic estimates recognizing the occurrence of heavily loaded coal-hauling vehicles put the design EAL at 2.697×10^7 (EWL = 8.63×10^8). All design EAL's represented projections for a 20-year period.

Overlay thickness designs also were determined for EAL levels presented above. Existing pavement conditions were defined as CBR of 5, an effective asphaltic concrete thickness of 3.30 inches, and the constructed (existing) 12.0 inches of crushed-stone base. The required overlay based on those parameters were:

Design EAL = 9.02×10^6
Asphaltic Concrete Overlay = 7.2 inches
Design EAL = 2.697×10^7
Asphaltic Concrete Overlay = 9.5 inches

The question of the remaining life for the existing structure remains. Using Kentucky flexible design curves and estimated in-place pavement conditions, an estimate of the remaining life for the existing structure can be determined. The existing pavement conditions were defined as:

CBR = 5
Effective Thickness of Asphaltic Concrete = 3.3 inches
Crushed-stone Thickness = 12.0 inches

TABLE 3. SUMMARY OF OVERLAY THICKNESS REQUIREMENTS
(DESIGN EAL = 7.47×10^6)

	SITE 1 STATION 59+00	SITE 2 STATION 56+40	AVERAGE
Mean Effective Thickness of Asphaltic Concrete (inches)	3.81	3.61	3.71
Standard Error (inches)	0.46	0.51	0.49
80th-Percentile Effective Thickness of Asphaltic Concrete (inches)	3.42	3.18	3.30
Mean Percent Deterioration	76.2	72.2	74.2
80th-Percentile Deterioration	68.5	63.7	66.1
Mean Recommended Overlay Thickness (inches)	6.54	5.21	5.88
80th-Percentile Recommended Overlay Thickness (inches)	7.20	5.81	6.50

The remaining fatigue life (EAL's) associated with those conditions was 1.8×10^4 . The remaining life was estimated by extrapolation of Kentucky flexible design curves to estimate the design fatigue level associated with 3.3 inches of asphaltic concrete over 12 inches of crushed stone at a design CBR of 5. Using that same analogy, the design life associated with the existing structure assuming no deterioration of the asphaltic concrete (i.e., 5 inches of asphaltic concrete on 12 inches of crushed stone) was estimated as 1.25×10^5 EAL's. This value is considerably different from the design EAL of 7.47×10^6 .

Reasons associated with such significant reductions in available fatigue life include:

- 1) a reduction of subgrade support from CBR 11 (design) to CBR 5 (in-place) and
- 2) variations associated with the use of the 1959 Flexible Design Curves versus the use of the 1981 480-ksi Flexible Design Curves for current analyses and evaluations. Using the 1981 design curves, the fatigue life associated with the existing 5 inches of asphaltic concrete on 12 inches of crushed stone was only 1.5×10^6 EAL's had the subgrade been a CBR 11. Using the same curves, the expected fatigue life associated with a CBR 5 subgrade is only 1.25×10^5 EAL's. Thus, a change of subgrade support from CBR 11 to CBR 5 results in a 90-percent reduction of available fatigue life.

The application of heavy loading also is very likely to result in early deterioration of asphaltic concrete. Deflection testing has indicated some degree of deterioration (effective asphaltic concrete thickness 3.3 inches versus the constructed 5.0 inches). The remaining fatigue life associated with the effective or "behavioral" structure (3.3 inches asphaltic concrete on 12 inches dense-graded aggregate) is 1.8×10^4 EAL's if the CBR is 5. The associated reduction of fatigue life is therefore 85 percent.

In interpreting the results, it should be noted the consumption of the available fatigue life does not indicate a rubbled pavement condition. It does, however, indicate that the limits of the elastic properties of the pavement have been equaled or exceeded and disintegration of the pavement structure may be anticipated with continued application of loadings.

While there has been considerable distress (in the form of deep rutting) at these specific locations, such is not the situation for the overall length of the section. It is at least possible to speculate that the observed occurrences represent localized areas of weakness. Thus, additional structural evaluations are recommended to determine the general extent of structural conditions throughout the section.

A more detailed discussion of variations associated with the two design procedures (1959 versus 1981) follows. It was assumed that the completed design, including 1 inch of surface, would have been 6 inches of asphaltic concrete on 12 inches of crushed stone (18 inches total pavement -- 33 percent asphaltic concrete). That structure represents the required pavement thickness design for a CBR 11 and 2.39×10^8 EWL's (7.47×10^6 EAL's) when using the 1959 Kentucky Flexible Design Curves (1). Using those same design curves and the same EWL (2.39×10^8 EWL's)

but a CBR 5 instead of CBR 11, the required pavement design is 7.5 inches of asphaltic concrete on 15 inches of crushed stone (22.5 inches total structure -- 33 percent asphaltic concrete). Thus, 4.5 inches of additional material (1-1/2 inches asphaltic concrete on 3 inches crushed stone) is required if the actual CBR is 5 instead of the anticipated CBR 11 on the basis of the 1959 design curves.

The 1981 Kentucky design curves were used for comparison. For a CBR 5 and 7.47×10^6 EAL's, the required pavement thickness is 25.75 inches (8.5 inches of asphaltic concrete on 17.25 inches of crushed stone). The required pavement thickness is only 21 inches (7 inches of asphaltic concrete on 14 inches of crushed stone) for CBR 11. Notice that the difference in thickness between CBR 5 and CBR 11 is very nearly the same as with the 1959 curves. However, the thickness requirements are much greater when using the 1981 curves, regardless of the design CBR. Using the 1981 design curves (480 ksi), the design was only worth a fatigue life of 2.5×10^6 EWL's if the CBR was 11 and only 4.0×10^5 EWL's if the CBR was 5.

Analysis of 1957 test data indicated that the required total pavement thickness should be 23 inches for a CBR 7 soil and 8×10^6 EAL's, as shown by the circled point marked "X" in Figure 14. Engineering judgment at that time indicated that the 2 inches were excessive. Thus, Curve X was positioned through 21 inches at CBR 7 superimposed as the solid line on Figure 14. The dashed line superimposed on Figure 14 illustrates where Curve X should have been positioned. Analysis made in 1968 indicated that Curve X should have been positioned at approximately 21.7 inches total thickness for a 33-percent asphaltic concrete pavement design.

Thus, use of the 1959 curves resulted in an underdesign for the projected fatigue level ($EWL = 2.39 \times 10^6$) when compared with designs obtained using the 1981 480-ksi design curves. This was further compounded by test data (laboratory and field) that indicated actual field conditions were very much inferior to anticipated design conditions (CBR 5 versus CBR 11). Overlay design calculations were determined using the 1981 flexible design curves (480 ksi) for three projected fatigue levels: 7.47×10^6 EAL's, 9.2×10^6 EAL's, and 2.697×10^7 EAL's. Overlay design calculations also incorporated the condition of the existing pavement structure using deflection data as well as laboratory CBR data. Recommended overlay thicknesses are presented in Table 4.

An accounting for the recommended overlay thickness for a design EAL of 7.47×10^6 is presented in Table 5. Three different factors affect the required overlay recommendation: deterioration of the in-place asphaltic concrete, variation in thickness design procedures (1981 curves require greater thicknesses than the 1959 curves), and the apparent inferior quality subgrade material (CBR 11 versus CBR 5).

The remaining 2.3 inches of asphaltic concrete overlay (Table 5) is required as substitution for the required 5 inches additional crushed stone that could not be placed since the pavement had already been constructed with 12 inches of crushed stone. An apparent asphalt-crushed stone substitution ratio is 2.17 inches of crushed stone per 1 inch of asphaltic concrete. From Table 5, it is apparent that only approximately 30 percent of the recommended overlay design may be attributed to variations among the two design systems. The remaining 70

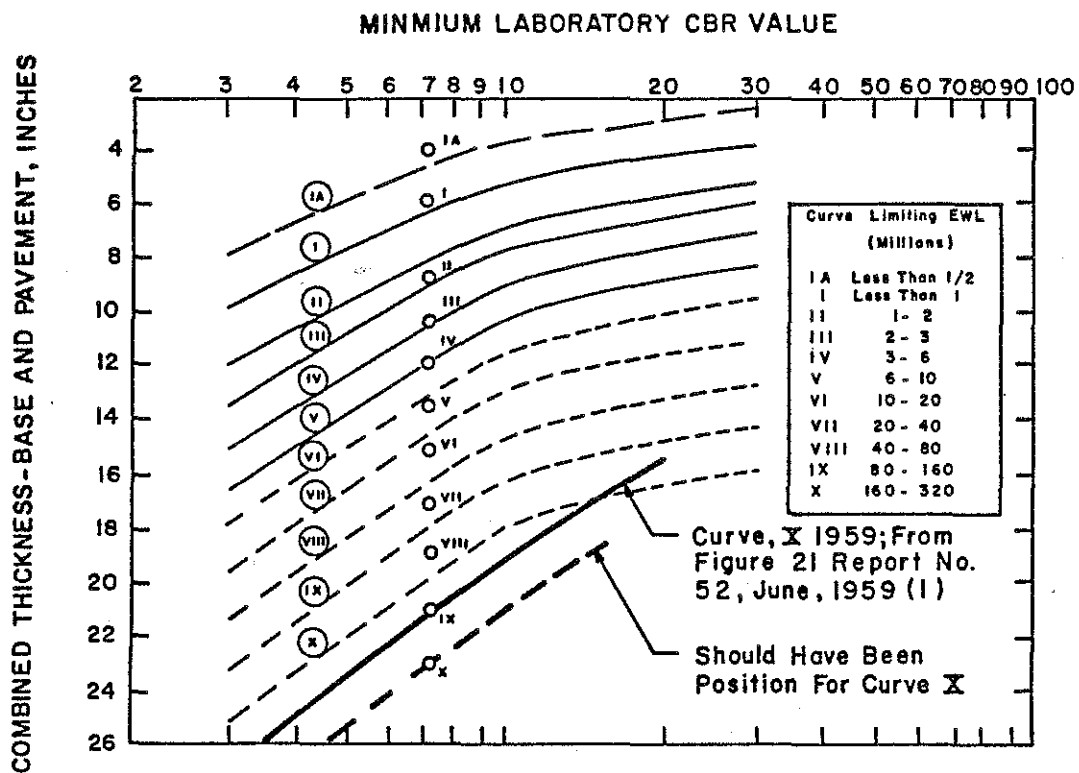


Figure 14. 1947 Design Chart Showing 1959 Revised Thicknesses for Each Median Point of the Traffic Volume Groups, Plotted at an Average CBR of 7.1 (1). Notes are Current.

TABLE 4. RECOMMENDED ASPHALTIC CONCRETE
OVERLAY THICKNESS

DESIGN 18-KIP AXLELOADS	OVERLAY THICKNESS (inches)
7.47×10^6	6.5
9.2×10^6	7.2
2.697×10^7	9.5

TABLE 5. ACCOUNTING FOR RECOMMENDED OVERLAY THICKNESSES
DESIGN EAL = 7.47×10^6

FACTOR AFFECTING OVERLAY THICKNESS	REQUIRED THICKNESSES		PERCENT OF RECOMMENDED OVERLAY THICKNESS
	ASPHALTIC CONCRETE (Inches)	CRUSHED STONE (Inches)	
Deterioration of Existing Structure	1.7		26.2
Variation Associated with Design Curves (1959 vs 1981)	1.0	2.0	29.6
Variation due to Inferior Subgrade Material (CBR 5 vs CBR 11)	1.5	3.0	44.2
Total	4.2	5.0	100.0
Recommended Overlay Design	6.5		
Difference	2.3		
Substitution Ratio: $50/2.3 = 2.17$			

percent must be attributed to deterioration (26.2 percent) and the change from a design CBR 11 to the in-place CBR 5 subgrade.

Similar analyses may be applied for designs for greater EAL levels. The proportion of the overlay attributed to deterioration will remain the same (1.7 inches). Variations relative to different CBR's and increased EAL levels will be specific to the position on the design curves.

LABORATORY TESTS

Bag samples of the subgrade were collected at both stations. At Station 59+00, the subgrade appeared to be a brown-to-gray sandy clay containing shale fragments (particularly under the inside lane). Under the outside lane, the subgrade gradually became a bluish-gray clay shale. The in-place moisture content was 7.3 percent. A slake-durability index test (18, 19) was performed on that material according to KM 64-513-79. The SDI was 32.5, which qualifies as soil-like material. By correlation, the CBR would be 2.1 (20). The material also was tested to determine the CBR value according to KM 64-501-80, and the final soaked CBR value was 4.0.

At Station 56+40, the subgrade was a gray clayey shale with chunky particles as large as 6 inches. The in situ moisture content was 6.5 percent. The slake durability index of the material was 81.8. This would classify as an intermediate material between rock and soil. A CBR test was not performed on the subgrade of this station. By correlation (20) the CBR would be 4.8.

A bag sample of the dense-graded aggregate also was collected at Station 54+40. The in situ moisture content was 3.1 percent. Perhaps some drying occurred between the time of exposure and processing in the laboratory.

DESIGNED AND AS-BUILT CAPABILITIES OF PAVEMENT

The design CBR was 11 (value credited to good-quality rock subgrade). The measured CBR at a point where some sandstone-derived material was present was 4.0. Material obviously inferior to that was found toward the outer wheel path, where the greatest rutting had occurred.

Not only was the pavement inadequate to carry the designed traffic, even if the final surface had been constructed prior to subjecting it to traffic, but the designed 1977 traffic (7.47×10^6 EWL's) was significantly less than 1983 estimates of EWL's (not considering actual weights of coal trucks). Considering coal trucks (estimate made by others), the 20-year 1983 design traffic would be 863×10^6 EWL's (27×10^6 EAL's) or 3.6 times greater than the 1977 estimate. Thus, if the subgrade had actually been rock or otherwise had equaled a CBR of 11, the pavement would have been designed to have a service life of 5.6 years instead of 20. By weighting coal trucks used in this analysis, the 1977 design traffic would be used up in 814 days of hauling.

SUMMARY

Plastic material (soil) was found at Station 59+00 where rock subgrade should have been.

Chunky, friable shale was found at Station 56+40, where rock subgrade should have been. According to past practices, high-quality rock roadbed material would have been credited with a CBR of 11. Shaly or lesser quality rock would have been credited with a CBR of 9. The Slake Durability Test would have disqualified the shale for service as rock subgrade material.

Overloaded trucks use up service life of a pavement at a much accelerated rate (21, 22, 23). There has been an occasion previously when the designed 20-year service life was used up in approximately 100 days.

Further field investigation and survey would be required to determine the extent of inadequate roadbed or pavement foundation on the project. Road Rater tests may suffice. Cores could be obtained for inspection of the materials. Deficiencies in structure may be deduced from Road Rater analyses, and additional depths (layers) of asphaltic concrete needed to strengthen the existing pavements to meet current requirements may be ascertained.

Shales are categorically untrustworthy under pavements. This has been proven repeatedly where so-called "hard shales" have been admitted. I 75 north of the Rockcastle River is a case in point. Cautious reliance on the Slake Durability Index is urged in the future, but never to qualify a shale as being equal to rock. On the other hand, shaly material may be rated proportionally (fractionally) to rock. Rock, on the other hand, should be rated more realistically in terms of and according to CBR. The use of 11 has been arbitrary and capricious.

The EWL-method of expressing weighted and equivalent axleloads is antiquated -- and should be abandoned. State and national engineers and principals chose the 18-kip equivalent axleload to be the basic unit after the AASHO Road Test (early 1960's). Casual reference in these respects is made to Research Study KYHPR-84-102, Estimation of Equivalent Axleloads, which is intended to define the probable error in estimating these important design parameters.

It is wrong to perpetuate a practice or constraint that fails to respect the reality of coal trucks and overloads in the design of highway pavements.

REFERENCES

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3. Memo to W. B. Drake, Assistant State Highway Engineer for Research, from J. H. Havens, Director of Research. Re: Rutting, Asphaltic Concrete Pavements, September 5, 1978. Research file P-3-1.
4. Memo to J. H. Havens, Director of Research, from D. C. Newberry, Research Engineer. Re: Rutting Investigations, I 64 and US 60, December 8, 1978. Research files P-3-1 and H-2-77.
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6. Allen, D. L., "Determination of Rutting in Asphaltic Concrete Pavements: Field Instrumentation and Laboratory Characterizations." Research Report 502, Division of Research, Kentucky Department of Transportation, August 1978.
7. Allen, D. L. and Deen, R. C., "Rutting Models for Asphaltic Concrete and Dense-Graded Aggregate from Repeated Load Tests." Proceedings, The Association of Asphalt Paving Technologists, 1980.
8. Allen, D. L., "A Computerized Analysis of Pavement Rutting." Research Report UKTRP-83-6, Transportation Research Program, University of Kentucky, February 1983.
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Research Program, University of Kentucky, January 1982. Prepared for the International Symposium on Bearing Capacity of Roads and Airfields, Trondheim, Norway, June 23-25, 1982.

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APPENDIX A

TRAFFIC PREDICTIONS, ESTIMATES OF
EWL's, AND PAVEMENT STRUCTURE DESIGNS

10-283

LAWRENCE CO. APD 231-48 Sta. 48 ± to Sta 162 ±

GIVEN: EWL = 863,000,000 → EAL = 26,968,750
 CBR = 7, 9, 11, 3 1959

REQD:

- ALT. 1 IF Designed Conventional Flexible
 ALT. 2 IF Designed Full Depth Bituminous
 ALT. 3 IF Pavement in place necessary overlay
 ALT. 4 IF DGA in place Bituminous Required

FEDERAL PROJECT - EAL greater than 5.0×10^6 ∴ 1959 CURVE
 Full Depth Bit = 2:1 CURVE XII

ALT. 1 & ALT. 2		ALT. 3		ALT. 4		ALT. 5	
CBR=3	CBR=7	CBR=9	CBR=11	CBR=3	CBR=7	CBR=9	CBR=11
Alt. 1	Alt. 2	Alt. 1	Alt. 2	Alt. 1	Alt. 2	Alt. 1	Alt. 2
19 3/4"	15 1/2"	14"	13 1/2"	19 3/4"	15 1/2"	14"	13 1/2"
8 3/4"	6 1/2"	6"	5 7/8"	8 3/4"	6 1/2"	6"	5 7/8"
1"	1"	1"	1"	1"	1"	1"	1"

Existing Pavement Design: DGA = 12 1/2" = 6 1/4 Bit.
 CBR = 11 (33%) B.B. = 5" = 5 Bit.
 B.G. = 1" = 1 Bit.
 12 1/4"

ALT. 3

	CBR=3	CBR=7	CBR=9	CBR=11
OVERLAY	7 1/2"	3"	1 3/4"	1"

ALT. 4

	CBR=3	CBR=7	CBR=9	CBR=11
BB	12 1/2"	8 1/4"	6 3/4"	6"
BS	1"	1"	1"	1"

6 - 1
 GENE DRAKE
 C.C. J.H. Harvans

TRAFFIC VOLUME GROUP 3000 +

COUNTY BOYD ROAD NAME CATTLETTSBURG - LOUISA ROUTE NO. US23

PROJECT LIMITS 0.3 MILE NORTH OF THE BOYO-LAWRENCE COUNTY LINE PROJECT NO. _____

LOADOMETER STATION REFERENCE * L06 (77) - ATR 42 (83)

- (1) Per Cent of Trucks 27
- (2) Average Axles per Truck 4.281
- (3) Average 24 Hour Traffic (1993 ADT) 8400
- (4) Average 24 Hour Truck Traffic = (1) x (3) _____
- (5) Average 24 Hour Truck Traffic at End of 20 Year Period = (1) x (3) 2,270
- (6) Average Axles per Truck at End of 20 Year Period = (2) x 0.19 4.471
- (7) Total Axles in 20 Years = (5) x (6) x 365 x 20 74,088,941

(A) Axle Load (Tons)	(B) Total Axles (7)	(C) % of Total Axles From Load Sta.	(D) Correction	(E) Corrected % of Total Axles (C) x (D)	(F) Total Axles by Weight Class (B) x (E)	(G) EWL Factor	(H) EWL for Two Directions (F) x (G)
4½-5½	74,088,941	10.000	0.09	10.090	7,475,574	1	7,475,574
5½-6½	"	3.203	0.13	3.333	2,469,384	2	4,938,768
6½-7½	"	3.118	0.27	3.388	2,510,133	4	10,040,532
7½-8½	"	6.619	0.15	6.769	5,015,080	8	40,120,640
8½-9½	"	9.114	0.11	9.224	6,833,964	16	109,343,424
9½-10½	"	9.337	0.05	9.387	6,954,729	32	222,551,328
10½-11½	"	4.647	0	4.647	3,442,913	64	220,346,432
11½-12½	"	2.618	0	2.618	1,939,648	128	248,274,944
TOTAL EWL for 20 year period (two directions)							863,091,642

10/11/83
 [Signature]

* INCLUDES COAL TRUCK WEIGHTS

TRAFFIC VOLUME GROUP 3000 \neq

COUNTY BOYD ROAD NAME CATTLETTSBURG - LOUISA ROUTE NO. US 23

PROJECT LIMITS 0.3 MILE NORTH OF THE BOYO-LAWRENCE COUNTY LINE PROJECT NO. _____

LOADOMETER STATION REFERENCE 1978-80-82 RURAL DATA- ATR 42 (83)

- (1) Per Cent of Trucks 27
- (2) Average Axles per Truck 4.281
- (3) Average 24 Hour Traffic (1993 ADT) 8400
- (4) Average 24 Hour Truck Traffic = (1) \times (3) 2270
- (5) Average 24 Hour Truck Traffic at End of 20 Year Period = (4) \times (3) 2270
- (6) Average Axles per Truck at End of 20 Year Period = (2) \neq 0.19 4.471
- (7) Total Axles in 20 Years = (5) \times (6) \times 365 \times 20 74,088,941

(A) Axle Load (Tons)	(B) Total Axles (7)	(C) % of Total Axles From Load Sta.	(D) Correction	(E) Corrected % of Total Axles (C) \neq (D)	(F) Total Axles by Weight Class (B) \times (E)	(G) EWL Factor	(H) EWL for Two Directions (F) \times (G)
4 $\frac{1}{2}$ -5 $\frac{1}{2}$	74,088,941	18.186	0.09	18.276	13,540,495	1	13,540,495
5 $\frac{1}{2}$ -6 $\frac{1}{2}$	"	9.991	0.13	10.121	7,498,542	2	14,997,084
6 $\frac{1}{2}$ -7 $\frac{1}{2}$	"	11.608	0.27	11.878	8,800,284	4	35,201,136
7 $\frac{1}{2}$ -8 $\frac{1}{2}$	"	11.922	0.15	12.072	8,944,017	8	71,552,136
8 $\frac{1}{2}$ -9 $\frac{1}{2}$	"	5.035	0.11	5.145	3,811,876	16	60,990,016
9 $\frac{1}{2}$ -10 $\frac{1}{2}$	"	1.619	0.05	1.669	1,236,544	32	39,569,408
10 $\frac{1}{2}$ -11 $\frac{1}{2}$	"	0.623	0	0.623	461,574	64	29,540,736
11 $\frac{1}{2}$ -12 $\frac{1}{2}$	"	0.244	0	0.244	180,777	128	23,139,456
TOTAL EWL for 20 year period (two directions)							288,530,467

10/11/83

KENTUCKY DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS
DIVISION OF DESIGN
REVISED
Pavement Design

TD 61-29A
Rev. 6-79

Sheet 1 of 4

County Lawrence Item 9-310.1 UPN FAP 064 0023 018-021 016 D

Road Name Louisa - Catlettsburg (US-23) F.P. 00APD 00537 021

From South of KY 3 near Louisa Bypass north to 0.2 south of Waller Branch
(Sta. 48+00± to Sta. 162+00±)

Traffic 3240 19 65 8220 1975
9100 1990 E.W.L. 239x10⁶ E.A.L. 7.47x10⁶

Existing: Type _____ Thickness _____ inches

Length 2.15 miles. Design Speed 60 M.P.H. Design CBR 11

FOR TYPICAL SECTION SEE ATTACHED SHEET

PAVEMENT

Alt 1

- 3" Compacted depth Crushed Sandstone Base (Cement Treated); OR
- 3" Compacted depth Dense Graded Aggregate Base; WITH
- 9" Standard Reinforced Portland Cement Concrete Pavement; OR
- 9" Non-Reinforced Portland Cement Concrete Pavement

Alt 2

- 12½" Compacted depth Crushed Sandstone Base (Cement Treated); OR
- 12½" Compacted depth Dense Graded Aggregate Base; WITH
- 5" Compacted depth Bituminous Concrete Base, Class S; OR
- 5" Compacted depth Bituminous Concrete Base; AND WITH
- 1" Compacted depth Bituminous Concrete Surface, Class A
- 0.80 lb/s.y. Bituminous Tack Coat

Alt 3

- 11½" Compacted depth Bituminous Concrete Base, Class S; OR
- 11½" Compacted depth Bituminous Concrete Base; WITH
- 1" Compacted depth Bituminous Concrete Surface, Class A
- 0.80 lb/s.y. Bituminous Tack Coat

*App. Tie To To
Project Construction
Sta. 71+00 to 162+00*

*Use Alt 2 on 3
for this tie in*

*E.F.R.
5-5-82
6-11-82*

E.F.R.

SUBMITTED E.B. Drake DATE 5/5/82 Asst. Dir., Division of Design
RECOMMENDED James B. Blair DATE 5/6/82 Director of Design
APPROVED W.K. Taylor DATE 5/10/82 Asst. State Highway Engineer
APPROVED _____ DATE _____ For Division Administrator FHWA

SHOULDER

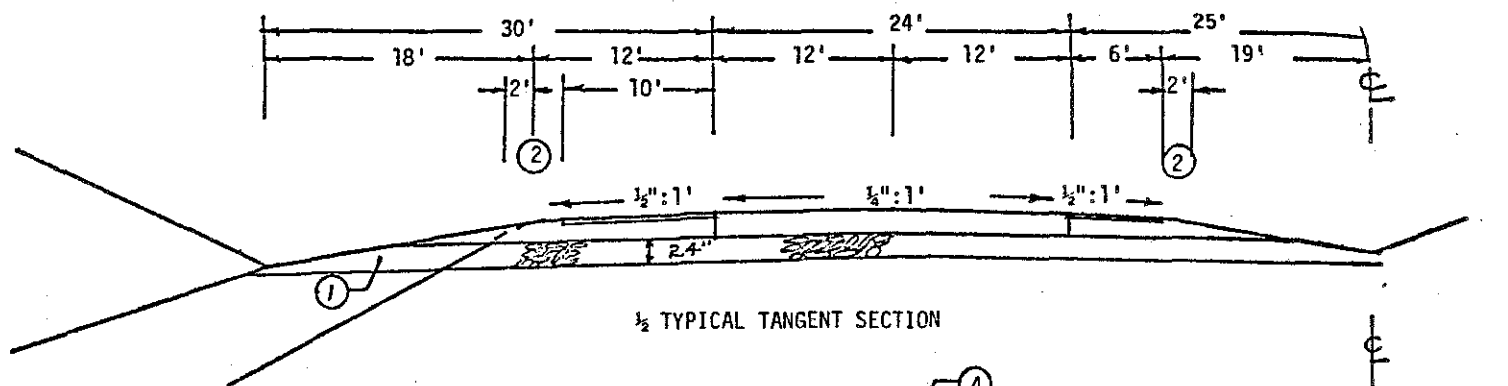
Alt 1, Alt 3

Full depth Compacted Crushed Sandstone Base (Cement Treated); OR
Full depth Compacted Dense Graded Aggregate Base; WITH
3" Compacted depth Bituminous Concrete Base Class S; OR
3" Compacted depth Bituminous Concrete Base; AND WITH
1" Compacted depth Bituminous Concrete Surface
0.80 lb/s.y. Bituminous Tack Coat

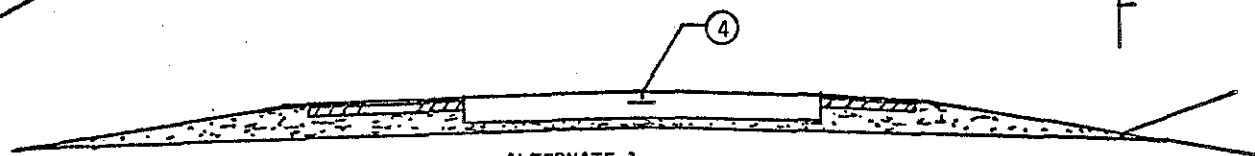
Alt 2

One Tift rock Roadbed Material; WITH
5" Compacted depth Crushed Sandstone Base (Cement Treated); OR
5" Compacted depth Dense Graded Aggregate Base; WITH
3" Compacted depth Bituminous Concrete Base Class S; OR
3" Compacted depth Bituminous Concrete Base; AND WITH
1" Compacted depth Bituminous Concrete Surface
0.08 lb/s.y. Bituminous Tack Coat
Bituminous Seal (All Alternates)
From outside edge of paved shoulder to a point two feet down the ditch or fill slope.
2.40 lb/s.y. Bituminous Prime Coat
20 lb/s.y. Crushed Aggregate Size 8

Plan Note No. 242 (Sandstone)
Plan Note No. 405
Plan Note No. 410
Plan Note No. 540
Plan Note No. 550
Special Provision No. 41A(79)
Special Provision No. 42A(79)
Special Provision No. 43B(79)
Special Provision No. 65(79)

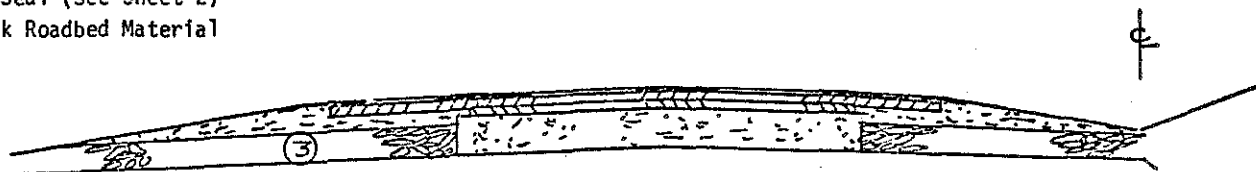


½ TYPICAL TANGENT SECTION



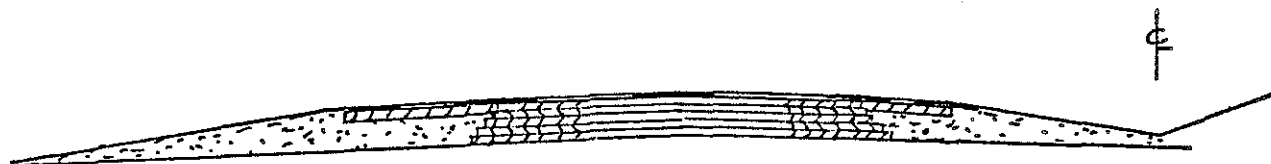
ALTERNATE 1

- ① Rock Roadbed
- ② Bituminous Seal (See Sheet 2)
- ③ Lift of Rock Roadbed Material



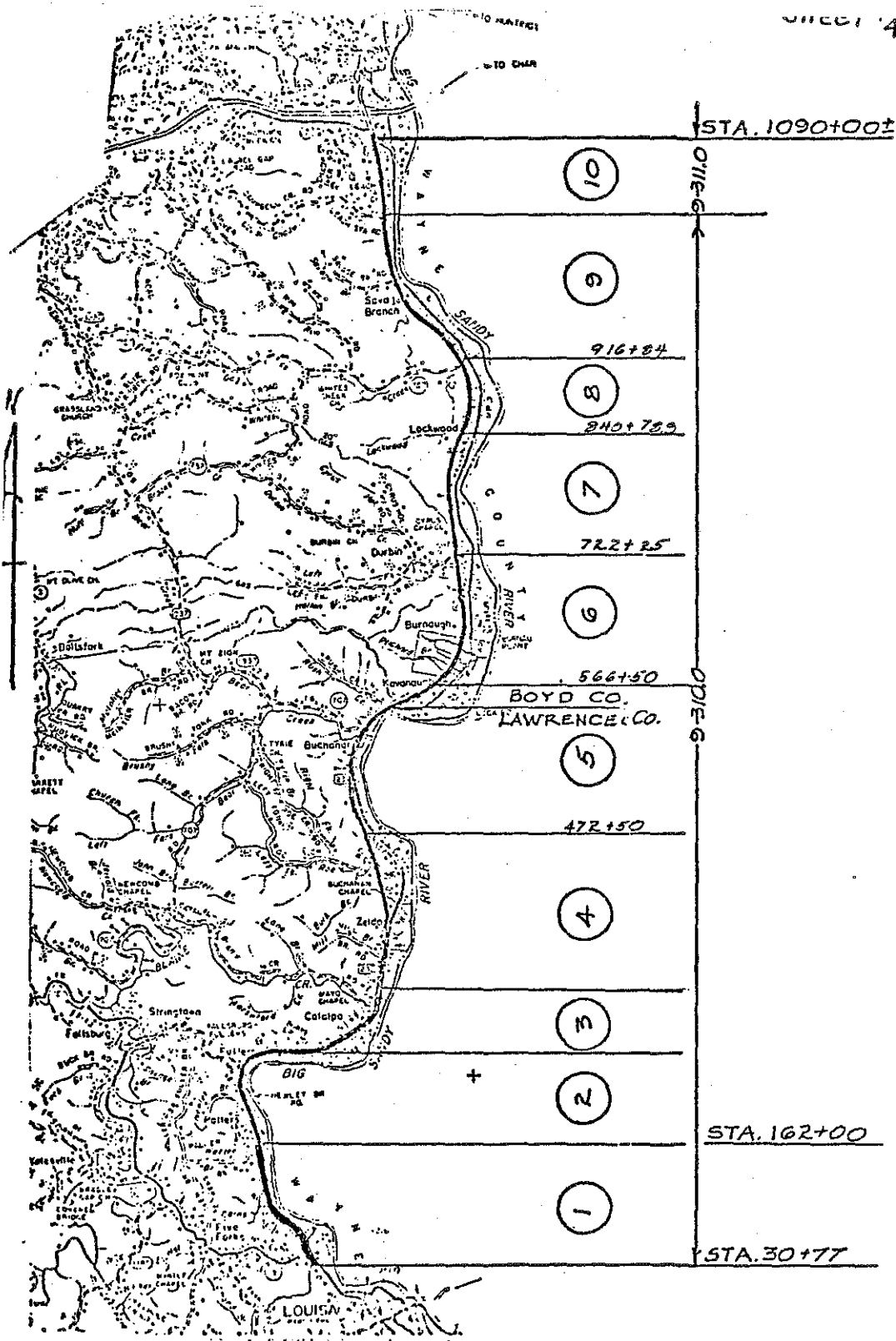
ALTERNATE 2

- ④ Longitudinal Sawed Joint



ALTERNATE 3

Sheet 4 of 4



KENTUCKY DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS
DIVISION OF DESIGN

TD 61-29A
Rev. 8-74

PAVEMENT DESIGN

REVISED

Sheet 1 of 6

County Boyd-Lawrence F.P. APD 537(21)
Road Name Louisa-Catlettsburg (US 23) S.P. AP 10-85-30L
AP 64-113-10L
From South of KY 3 (Sta. 30+77+) North of Louisa

To KY 3 just south of the I 64-US 23 Interchange.

Traffic: 3240 , 19 65 , 8220 75
9100 , 19 90 . E.W.L. 239 Million
Existing: Type _____ Thickness _____ inches
Length 23.18 miles. Design CBR See below.

Item No. 9-310.2, 3, 4, 5, 6 FOR TYPICAL SECTION SEE ATTACHED SHEET

CBR = 7 (1) 12.71 miles

Item No. 9-310.7, 8, 9,

CBR = 2 (2) 7.10 miles

Item No. 9-311.0, 9-310.1

CBR = 11 (1) 3.37 miles

- (1) Rock subgrade increase bottom 4" of the DGA Base 10% by weight, including shoulders. See Attached Notes.
- (2) The top 6" of subgrade under the DGA Base shall be soil cement containing 10% by weight portland cement. *No Subgrade Note Required*

NOTE: This revision applies to all sections not let to contract.

(E.F.R.)

9-7-78
1-24-79

RE-SUBMITTED E.B. Porch (See attached sheets for Pavement Designs) DATE 9/7/78 Asst. Dir., Division of Design
RECOMMENDED [Signature] DATE 9/7/78 Director of Design
APPROVED E.K. [Signature] DATE 9/8/78 Asst. State Highway Engineer
APPROVED _____ DATE _____ For Division Engineer FHWA

Item 9-310.2, 3, 4, 5, 6

PAVEMENT (Stage 1)

- 13" Compacted Depth Dense Graded Aggregate Base
- 6½" Compacted Depth Bituminous Concrete Base (2-3½" Courses)
- *1" Compacted Depth Bituminous Concrete Surface, Type B
- 0.80 lbs./s.y. Bituminous Tack Coat

Item 9-310.7, 8, 9

PAVEMENT (Stage 1)

- 12½" Compacted Depth Dense Graded Aggregate Base
- 9½" Compacted Depth Bituminous Concrete Base (3½"+3"+3" Courses)
- *1" Compacted Depth Bituminous Concrete Surface, Type B
- 0.80 lbs./s.y. Bituminous Tack Coat

Item 9-310.1 & 9-311.0

PAVEMENT (Stage 1)

- 12" Compacted Depth Dense Graded Aggregate Base
- 5½" Compacted Depth Bituminous Concrete Base (2-2 3/4" Courses)
- *1" Compacted Depth Bituminous Concrete Surface, Type B
- 0.80 lbs./s.y. Bituminous Tack Coat

Item 9-310.1, 2, 3, 4, 5, 6, 7, 8, 9 & 9-311.0

SHOULDERS (Stage 1) All Sections

- 11" Compacted Depth Dense Graded Aggregate Base
- 2.80 lbs./s.y. Bituminous Prime Coat
- Bituminous Seal
- 2.40 lbs./s.y. Bituminous Seal Coat
- 70 lbs./s.y. Crushed Aggregate Size No. 78
- 2.80 lbs./s.y. Bituminous Seal Coat
- 15 lbs./s.y. Crushed Aggregate Size No. 8
- **2.00 lbs./s.y. Bituminous Seal Coat
- **20 lbs./s.y. Crushed Aggregate Size No. 8

Item 9-310.1, 2, 3, 4, 5, 6, 7, 8, 9, & 9-311.0

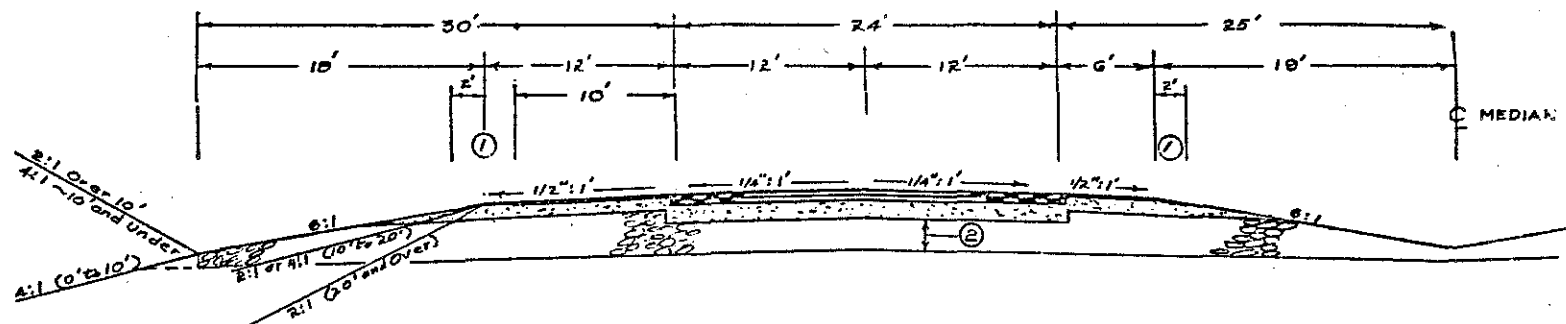
OVERLAY (Stage 2)

- Include sufficient tonnage of Bituminous Concrete Surface for Leveling.
- An acceptable high type skid resistant wearing course
- 0.80 lbs./s.y. Bituminous Tack Coat

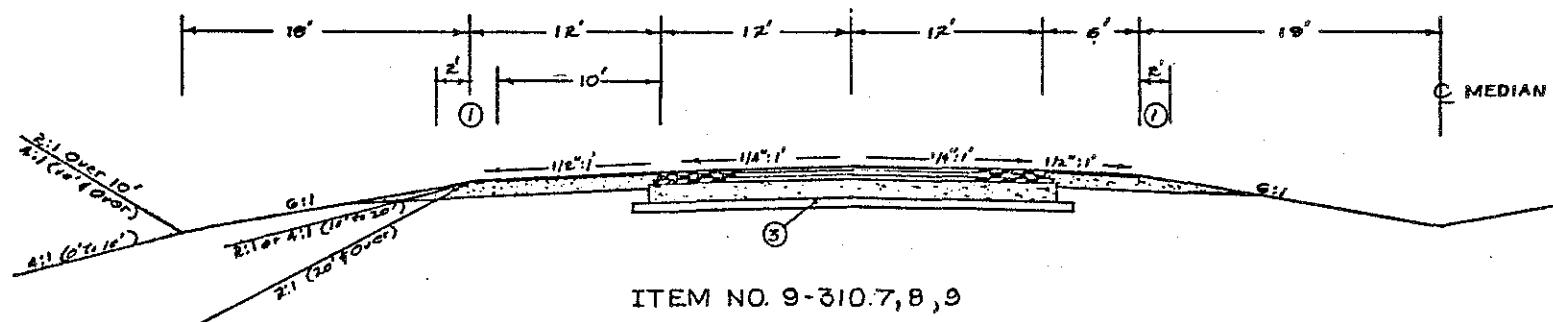
Bituminous Material for Bituminous Concrete shall be AC-20.

*Non-polishing sand required (Plan Note 425).

**The last application of oil and aggregate shall extend throughout the shoulder and two foot down the ditch or fill slope to retard vegetation growth and help prevent erosion.



ITEM NO. 9-310.1, 2, 3, 4, 5, 6, & 9-311.0

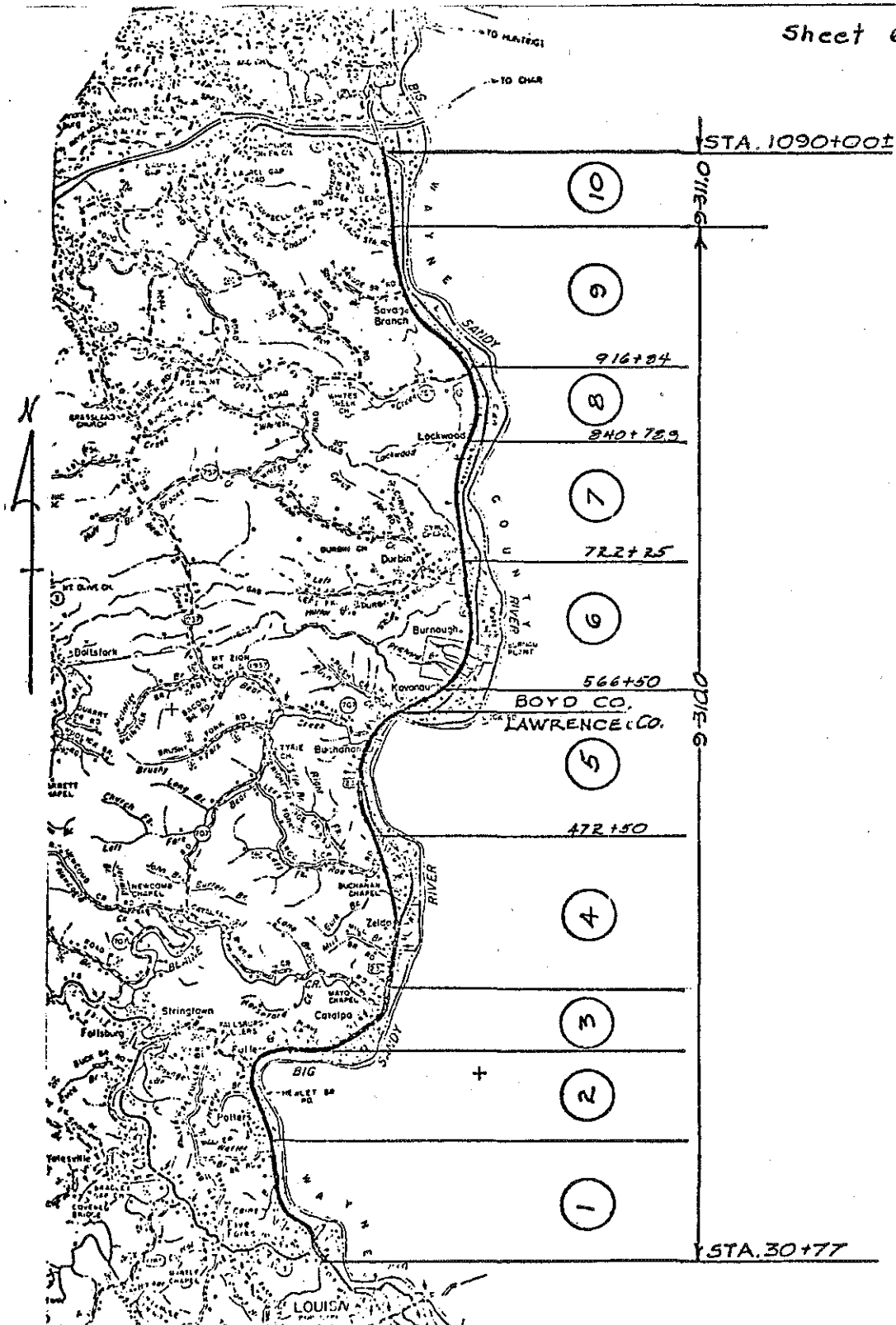


ITEM NO. 9-310.7,8,9

- ① See ** on Sheet No. 2
- ② Rock Subgrade. See attached notes
- ③ Soil Cement. See note (2) on Sheet No. 1

1/2 TYPICAL TANGENT SECTION
4 LANE ~50' DEPRESSED MEDIAN

Sheet 3 of 6



M E M O R A N D U M

TO: G. R. Best
Division of Design

FROM: H. A. Mathis, P.E. *HAM*
Assistant Director
Division of Materials
Geotechnical Section

DATE: March 3, 1978

SUBJECT: Lawrence County
APD 537 (21); AP 64-113-10L
Louisa - Catlettsburg Road (US 23)
Section I
Stations 48+00 to 129+00

We have reviewed the Geotechnical Engineering Report and soil profile submitted by the soil consultant and approve them.

The consultant recommends rock subgrade for the project. There is sufficient sandstone and/or siltstone on the project, see the attached "Summary of Solid Rock", to require rock roadbed construction in accordance with Section 204.10.01 of the Standard Specifications for Road and Bridge Construction.

A CBR design value of 11.0 is estimated for this type of subgrade.

HAM:ks

cc: E. Rassenfoss
W. McKenzie

Attachment

SUMMARY OF SOLID ROCK - NEEDED AND AVAILABLE

LAWRENCE CO. APD 537(21) LOUISA-CATLETTSBURG ROAD (U.S. 23) CONSTRUCTION SECTION -1

STA. TO STA.	S.R. NEED 2' R.S.	S.R. NEED 1' R.S.	ROCK FILL NEED		AVAILABLE TYPE III (under) MATERIAL	REMARKS
31+00 - 40+00	13.221	8.955	-		-	
40+00 - 70+00	44.070	29.850	46.155		-	
70+00 - 100+00	44.070	29.850	16.389		*316.674	*EXCLUDES ALL SHALES
100+00 - 130+00	44.070	29.850	49.237		*141.743	*EXCLUDES ALL SHALES
130+00 - 160+00	23.137	15.671	-			
160+00 - 162+00	1.469	995	-			
DETOURS	6.767	4.584	-			
EMB. BENCH BACKFILL			26.028			
TOTAL	176.804	119.755	139.809		458.417	
SUMMARY:			NEEDED FOR 2' ROCK SUBGRADE = 176.804 C.Y. NEEDED FOR SELECT ROCK FILL = 139.809 C.Y. TOTAL NEED = 316.613 C.Y. AVAILABLE TYPE III EXCLUDING SHALES = 458.417 C.Y.			

APPENDIX B

MEMO REPORT TO W. B. DRAKE, ASSISTANT STATE
HIGHWAY ENGINEER FOR RESEARCH, FROM J. H. HAVENS,
DIRECTOR OF RESEARCH, SEPTEMBER 5, 1978;
Re: RUTTING; ASPHALTIC CONCRETE PAVEMENTS (FILE P-3-1)

September 5, 1978

P-3-1

MEMO TO: W. B. Drake
Assistant State Highway Engineer
for Research

FROM: Jas. H. Havens
Director of Research *J. H. Havens*

SUBJECT: Rutting; Asphaltic Concrete Pavements.

On August 23, a cross section of full-depth, asphaltic concrete pavement, rather deeply rutted, was exposed by trenching across the lane. The site was on the hill, westward from the Hyden Spur toll plaza, on the Daniel Boone Parkway. The trench traversed the truck lane only. The exact location was M.P. 42 + 896 feet.

Theretofore (1957), trenching had disclosed the deformation to extend through the granular base course and into the subgrade soil. In fact, only about 15% of the depression at the surface was attributed then* to the asphaltic concrete layer. A full-depth

*Research Report 135; 1959
pavement had not been trenched before. Some instances of deep rutting had been studied, and some had been cored. One was on US 23 in front of Ashland Oil Company's research building. A full-depth pavement built about 1940, rebuilt in 1970, never developed noticeable rutting. There, slag used as traffic-bound surface before paving had cemented somewhat and formed a rigid crust under the asphaltic concrete. Rutting had been observed on the Watterson Expressway in Louisville and a site was trenched in 1957. The beltline, northeast of Lexington rutted somewhat and was trenched about 1957. Rutting on I 75, northward from the Rockcastle River (hill about M.P. 51) developed rutting about 1967, or before. Several measurements were made there. Unfortunately, it was not trenched. The wheel tracks were leveled with blacktop on one or more occasions, and a full-width overlay was placed in 1976. Later, 14 + miles were milled and re-surfaced. Rutting was only contributory to the project.

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Shoving may accompany rutting. There is no way to discern shear movement downgrade (due to wheel traction) unless lines are scored into the surface across the wheel tracks. We have been remiss in the past in not scribing lines for this purpose at several places of interest.

Detection and characterization of rutting is important in pavement design criteria and in pavement management strategies. Wear due to studded tires must be detected and isolated from rutting.

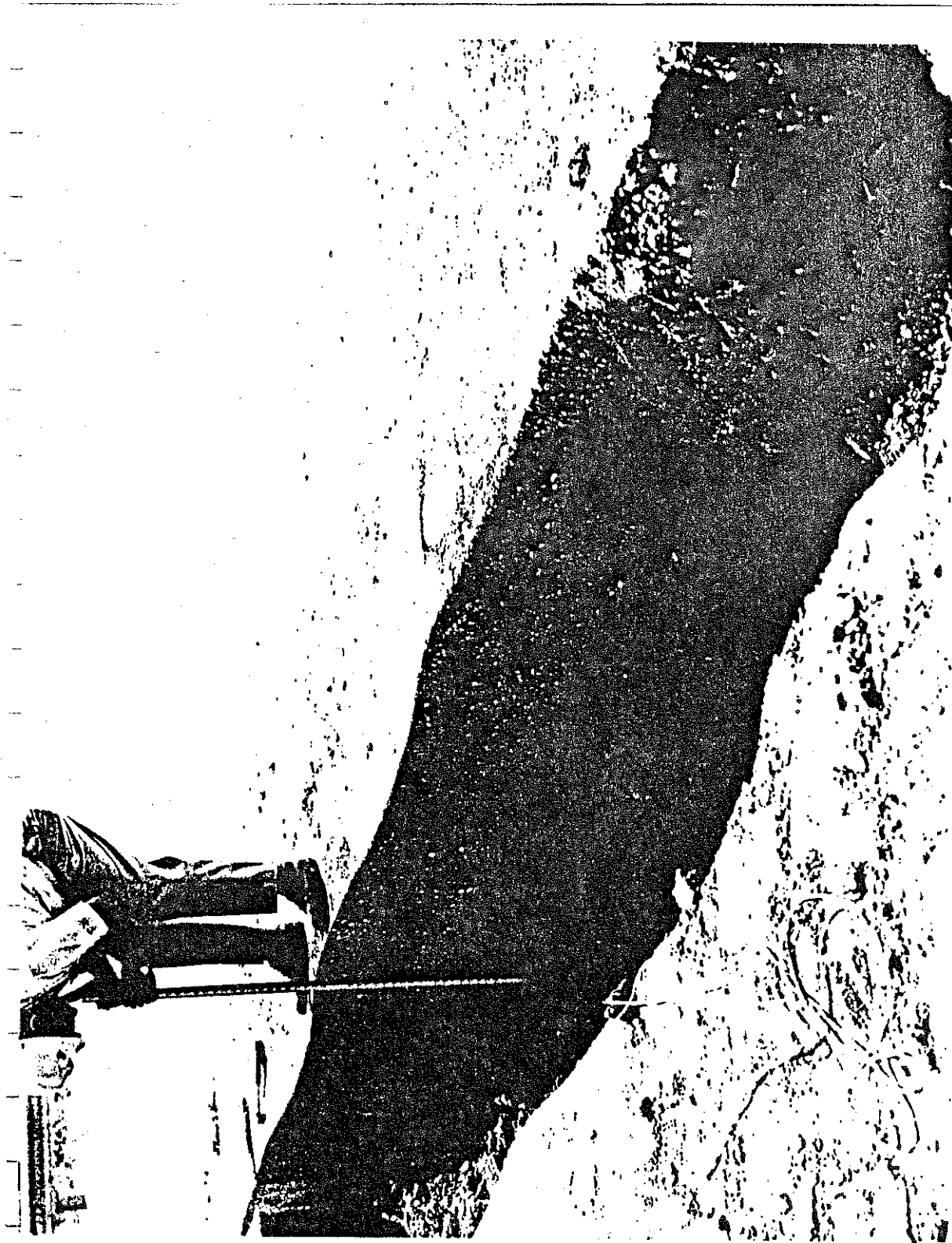
The designed structure on the Parkway was 17 inches of asphaltic concrete on 2 feet of rock subgrade (intended to be largely sandstone from the excavations). The section was opened to traffic 9-1975. Heavy, coal traffic has been on the road continually. AC 10 was used in asphaltic concrete base. A copy of the mixture designs is attached.

Several photographs are attached for the record and for the benefit of others who did not witness the exposure.

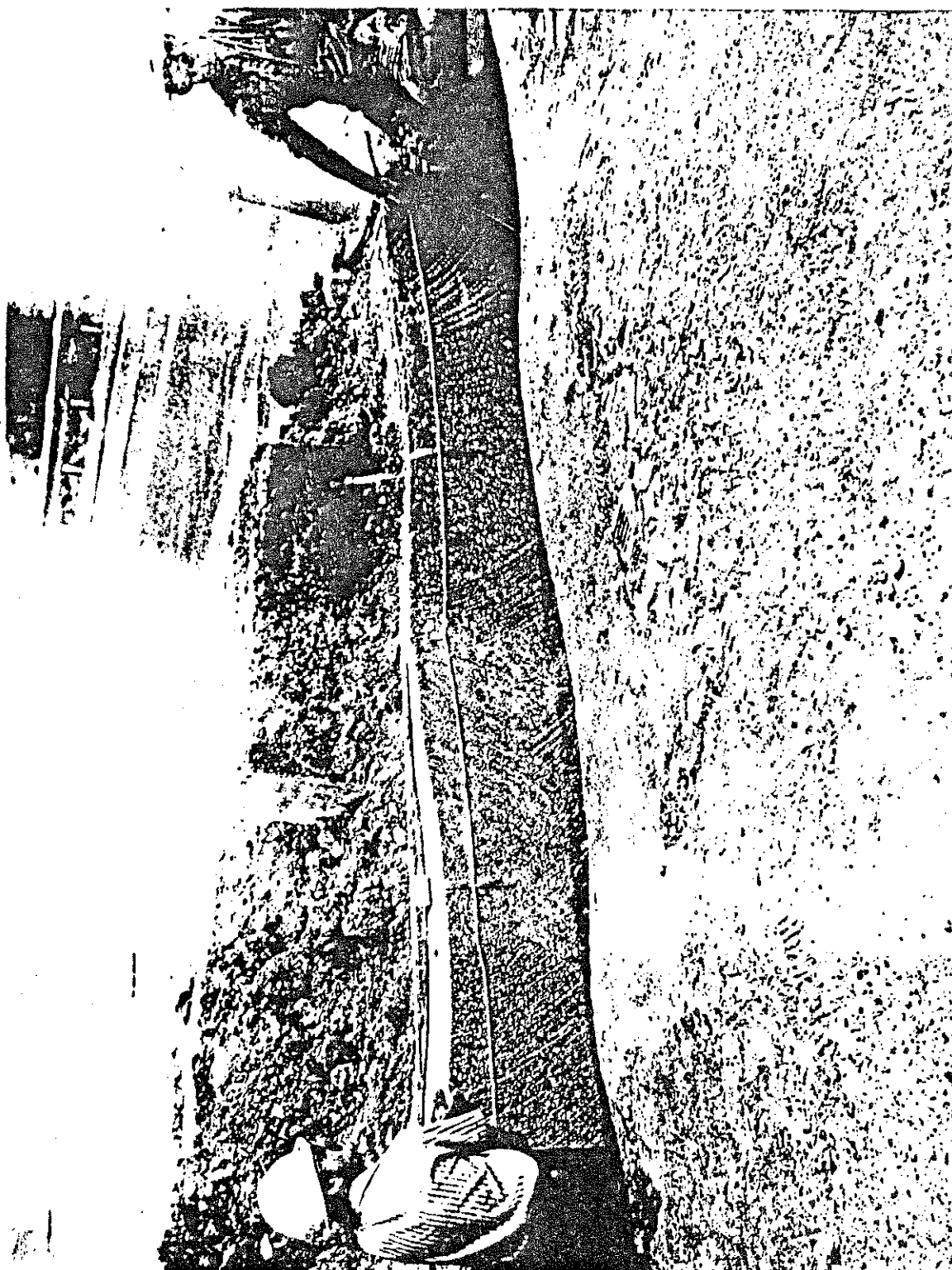
There was no apparent deformation (rutting) in the subgrade. This means that the bituminous concrete was adequate to protect the subgrade material -- even under very heavy wheel loads. Only the upper two to three inches of the asphaltic concrete was overstressed. Only the uppermost layers are subjected to high temperatures (summer sun) (have been known to reach 156°F). Asphaltic concrete farther at depth proved altogether worthy of heavy-duty service there. The rock subgrade was not as "rocky" as expected. Road Rater stiffness tests indicated greater elastic response in the wheel paths than elsewhere.

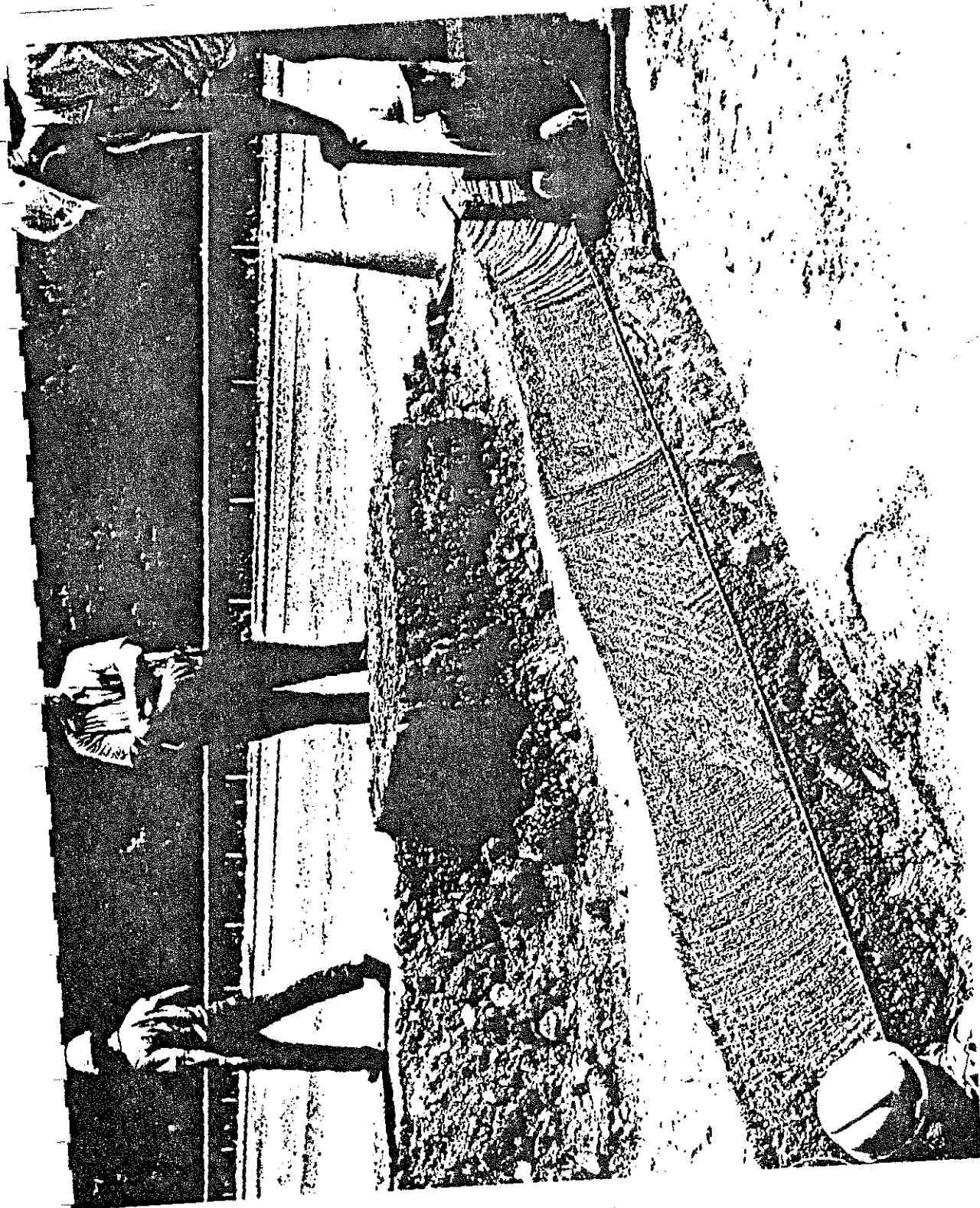
gd
Attachments



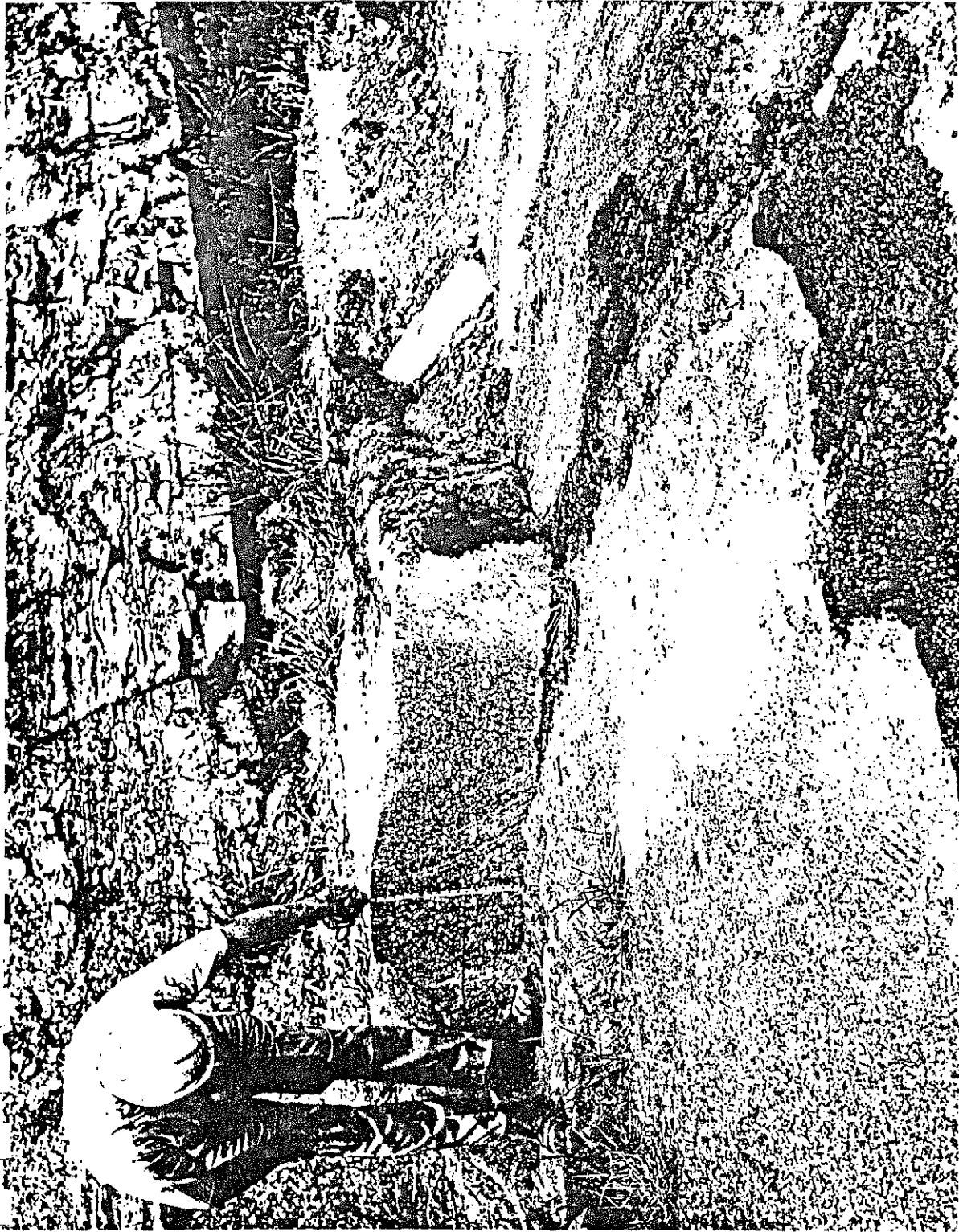


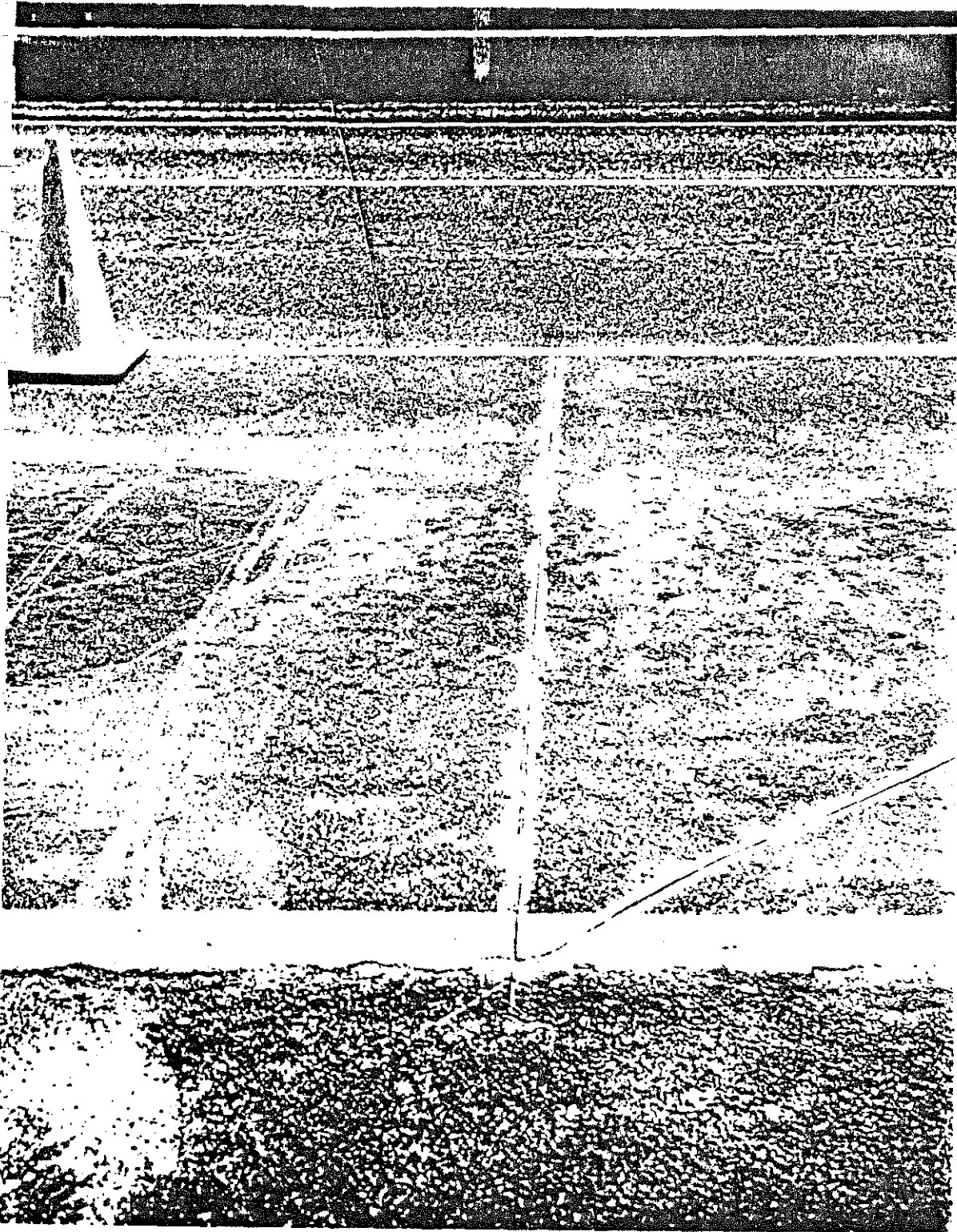












KENTUCKY DEPARTMENT OF HIGHWAYS

Laboratory Mix Design Report for a Bituminous Mixture

County Clay-Leslie Project No. KBP 11 SP 76-130
SP 66-22
 Name of Highway _____
 Laboratory No. _____ Date Received 7/13/73
 Identification Class "I" Base Date Reported 7/27/73
 Submitted by Ray Gilbert
 Paving Contractor and Location Nally & Gibson

DESIGN DATA AND RESULTS

Aggregate (Type & Size)	Source & Location	Percentage
#57 Limestone	Nally & Haydon, Harlan, Kentucky	60%
Limestone	Nally & Haydon, Harlan, Kentucky	40%

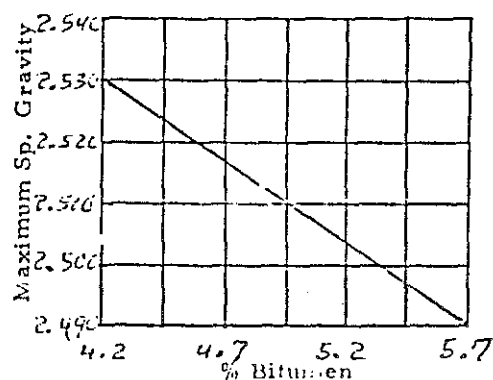
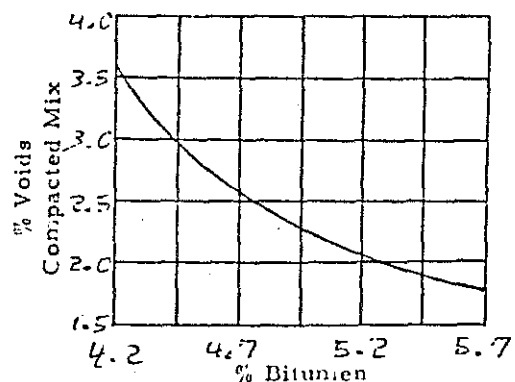
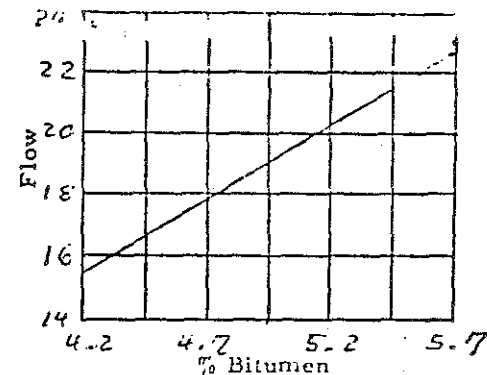
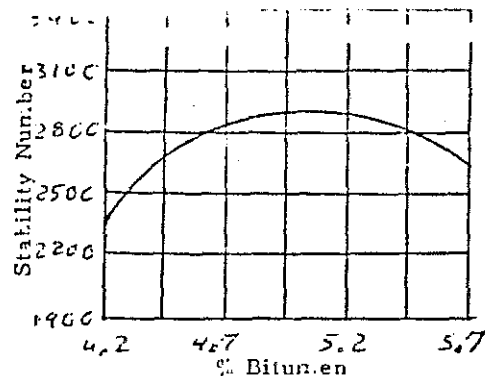
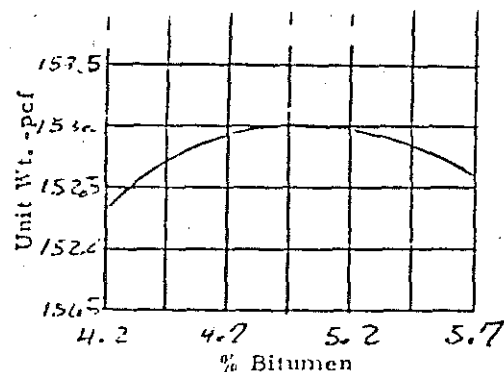
Bitumen (Type & Grade) AC-10 Source Ashland Oil # Ashland

Recommended Mixing Temperature* 295 ± 10 °F

*This temperature indicated from kinematic viscosity of bitumen for normal conditions.

Compaction: 50 Blows, for Medium Traffic Inten.

Sieve Size	Stockpile Aggregate Gradations					Design Gradat
	57 Stone	Limestone Sand				
1"	100	100			86 - 100	100
1/2"	51	100			40 - 60	71
#4	3.5	95			50 - 90	40
#8	1.1	74			25 - 75	30
#16	0.7	50			15 - 50	20
#50	0.5	21			5 - 25	5
#100	0.5	12			3 - 15	5
#200	0.4	7				



Design Results at Optimum

Unit Weight	153.0 pcf
Stability	2900 lbs
Flow	19 ins
Voids in Mix	2.3%
Max. Sp. Gravity	2.509

Note: The design bitumen content is recommended for the design mix gradation shown. Deviations from the materials furnished the laboratory or in the actual job gradation may require an adjustment in the design bitumen content; however, every effort should be made to produce a mix according to the design. Maximum Specific Gravity results may be used for density calculations.

Remarks: ~~This project is full depth asphalt construction. Use a bitumen content of 5.5 in the first coarse and 5.0% in all following coarses. Design results listed above are for a bitumen content of 5.0%.~~
Maximum specific gravity at 5.5% bitumen is 2.496.

opies to: Ray Gilbert

Respectfully submitted,
John McChand
Director of Materials

Res. Ant. Reel 908

HD 64-404
Rev. 2/69

ASPHALT PLANT (BATCH) MIX DESIGN REPORT

Date 8-6-72
COUNTY Clay-Leslie Project No. BBP 14-0
Contractor E. J. & Gibson Plant Location Hydon
Class "I" Base Lbs./batch 6,000

	Name and Location of Aggregate Source	Type	Size	Percent
COARSE	Hydon-Hydon, Ky	L. Stone	# 57	10
FINE	" " "	"	8 nd	10
FILLER				

PLANT INFORMATION

Aggregate/Batch 5,670 lbs.
Asphalt/Batch 330 lbs.
TOTAL BATCH WEIGHT 6,000 lbs.
Gallons of Asphalt/Batch
(Fluidometer) _____ gal.

Bin No.	Percent	Pounds	Accumul. Wt.
4	50	1701	1701
3	26	1474	3175
2	8	454	3629
1 (sand)	34	1928	5557
Filler	2	113	5670
TOTALS	100	5670	xxxxxxxxxxxx

ASPHALT INFORMATION

Type AC Grade 10
Producer Ashland
Percent Bitumen (by Design) 5.5
Field Extraction Tests: 5.5 5.5 5.5
Temp. Bitum. Mix 285

COLD FEED INFORMATION

Natural Sand _____ %
(checked at cold feed)
Cold Feed Settings: _____

PLANT SCREENS

1 1/2 " Scalper 1/4 "
2/10 " 3/10 "

SIEVE SIZE	JOB MIX FORMULA	COMBINED MIX GRADATION			Stockpile Grading of COARSE Aggr.	Stockpile Grading NATURAL	Grading DIFF.
1 1/2"		BTN	EXT	100	100		
1"		100	100	99	100		
3/4"							
1/2"	63-75	67	68	67	27		
3/8"							100
No. 4		44	40	43	1.0		100
No. 8	28-36	35	29	31	1.0		
No. 16		27	20	20			
No. 50		12	7	1			
No. 100	3-9	8	3	3.7			
No. 200							

REMARKS: ASPHALT COVERED TO 5.2%, MIX IS FINER ON # 10 & 20 SIEVES THAN MARSHALL MIX DESIGN.

cc: District Materials Engineer
Bituminous Section of Central Lab.
Central Laboratory File

Project Engineer E. J. & Gibson
Plant Inspector E. J. & Gibson
Materials Rep's Clayton, Clinton, Smith
Raney

CLAY - LESLIE DSP-A
CLASS I BASE

9/20/73

$$G_o = \frac{100}{\frac{95}{2.72} + \frac{5}{1}} = 2505$$

$$G_{AGG} = 2.72$$

$$G_s = 2.452$$

$$W_{AGG} = .95 \times 245.2 = 232.74$$

$$W_{ASPH} = .05 \times 245.2 = 12.26$$

$$W_{ABS} = \frac{17.26}{17.26}$$

$$\% EFF. ASPH = \frac{17.26}{245.2} = .05 \quad 5\%$$

$$VOL. AGG = \frac{232.74}{2.72} = 85.64$$

$$VOL. ASPH = \frac{12.26}{1} = 17.76$$

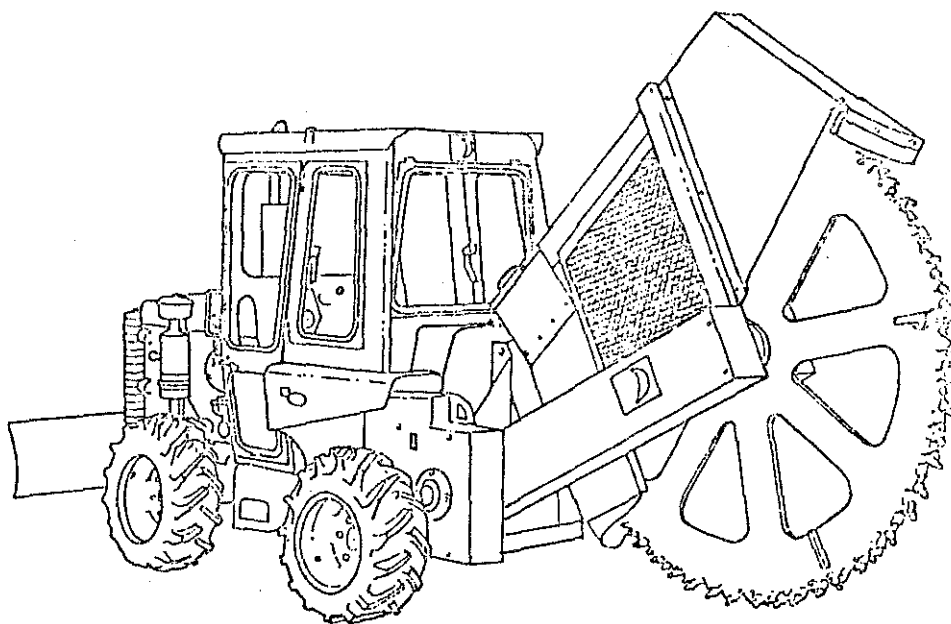
$$97.90$$

$$VOL. A = 100 - 85.64 = 14.36\%$$

$$\% AIR = 100 - 97.90 = (2.1\%)$$

100-1000
100-1000
100-1000

100-1000
100-1000
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One of the new items from Ditch Witch...

The DITCH WITCH EARTH SAW CUTS TRENCHES IN FROZEN EARTH, ROCK, CORAL and most types of roadways!

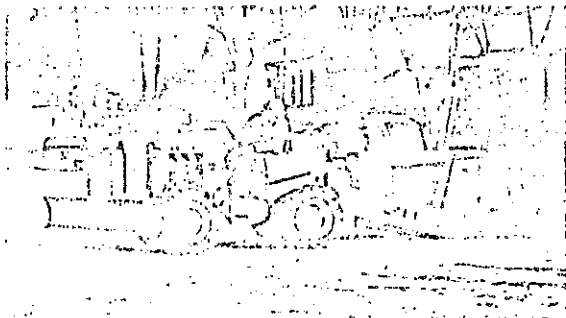
DITCH WITCH Earth Saw, the year-round trencher, cuts trench where you couldn't before—in frozen earth, rock, coral and most types of roadways.

DITCH WITCH Earth Saw attachments enable you to trench under conditions where previously it was either impractical or impossible. Two models are available to provide cuts to either 24 or 30 inch depths and 4 inch widths. The Earth Saw attachment replaces the regular digging assembly attaching with four bolts to R65 and R60 DITCH WITCH trenchers.

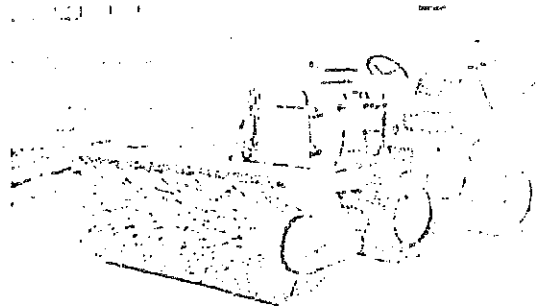
The Saw is mechanically powered for maximum performance while vehicle travel is independently controlled by hydraulic drive

system. Special carbide teeth revolve during trenching, keeping teeth sharp and insuring maximum tooth life. Four-wheel drive rubber-tire mounting of the basic vehicle provides great maneuverability. The Earth Saw lets you spread equipment costs, too. If you already have an R55 or R60, the initial investment is out of the way. In the winter, buy an Earth Saw and wait until warm weather to purchase the regular digging assembly. Either way, you get more production and stretch your equipment dollars. Ask your DITCH WITCH Professional for a free demonstration at your job site. You will see another idea that makes DITCH WITCH THE LEADER in underground service construction equipment.

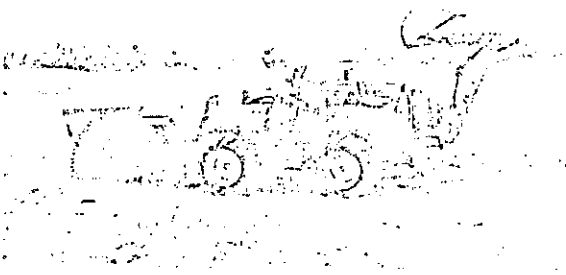




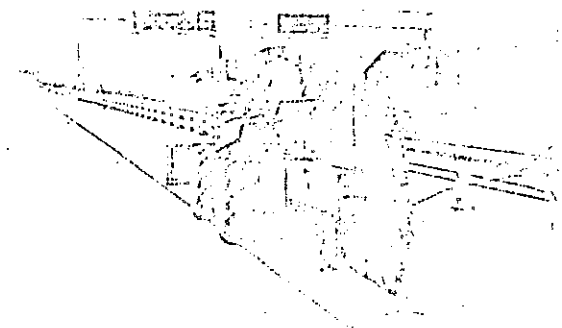
Ditch Witch Earth Saw cuts through completely frozen ground service installations.



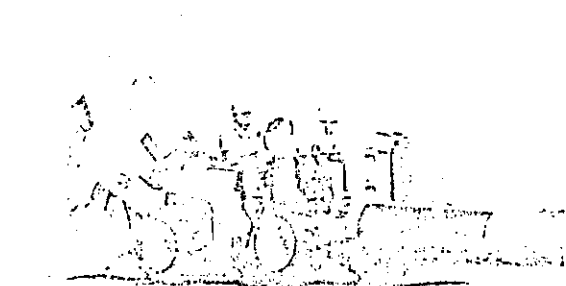
Cleanup on roadway jobs is quick and easy with the new clean-sweep broom attachment.



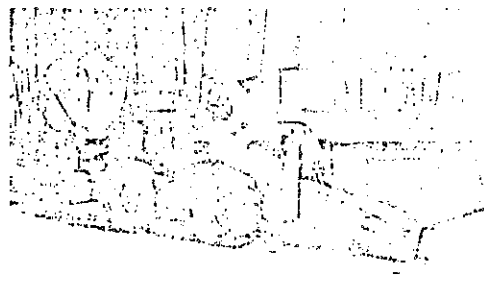
The Earth Saw cuts through coral.



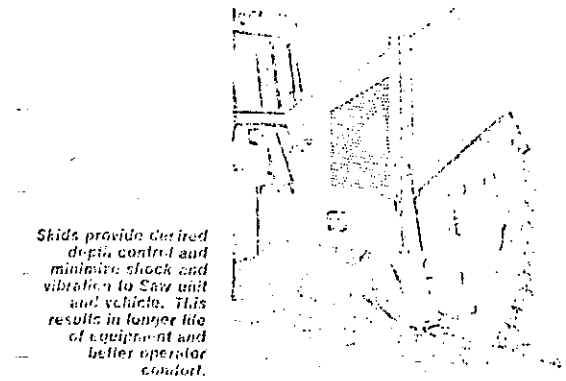
Installing conduit in stabilized asphalt base is quick and easy with the Earth Saw.



Compact with rubber-tire mounting for maneuverability. Rugged construction for dependability and long service.



The Earth Saw cuts through solid rock for telephone service line.



Skids provide desired depth control and minimize shock and vibration to Saw unit and vehicle. This results in longer life of equipment and better operator comfort.



Special carbide teeth revolve during trenching for maximum digging performance and tooth life.

DITCH WITCH EARTH SAW

ENGINE VEHICLE

Ditch Witch trencher vehicle; Model ES24, 4-wheel drive, rubber tire mounted, power steering and power options as follows: Wisconsin 177 CID gasoline, 65 HP industrial air-cooled engine; Ford 192 CID gasoline, 58-HP industrial water-cooled engine; Ford 192 CID diesel, 53-HP industrial water-cooled engine.

CUTTING TOOTH TYPE

Rotating, self-sharpening conical bit with spring retainer.

CUTTING TOOTH ADAPTATION

The carbide cap tooth is available for increased tooth life in relatively soft formations. A carbide insert tooth is available for faster cutting speeds and for use in relatively hard formations.

DRIVE SHAFT TYPE

Roller chain drive completely shielded for safety and protection from dirt.

CHUCK TYPE Driver Sprocket On Trencher to Reduction Shaft

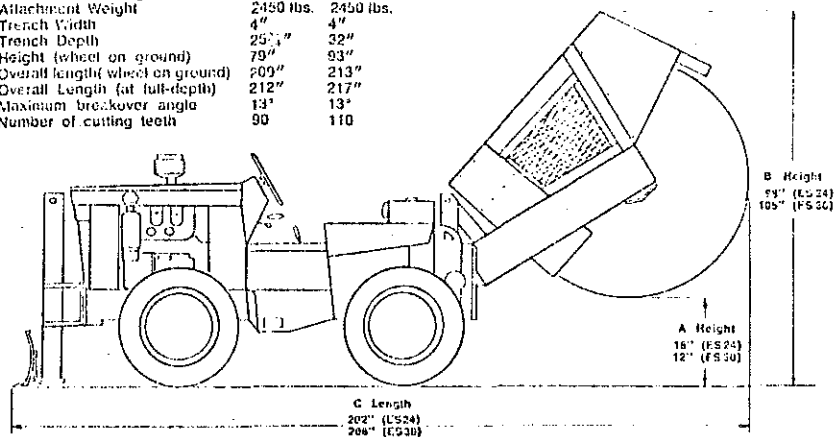
1 1/2" pitch roller chain

VEHICLE DIMENSIONS

	MODEL ES24	MODEL ES30
A-Ground clearance at wheel (wheel raised)	18"	12"
B-Maximum height (wheel raised)	98"	105"
C-Overall length including trencher (wheel raised)	202"	208"
Basic Vehicle Weight	4100 lbs.	4100 lbs.
Attachment Weight	2450 lbs.	2450 lbs.
Trench Width	4"	4"
Trench Depth	25 1/4"	32"
Height (wheel on ground)	79"	93"
Overall length (wheel on ground)	209"	213"
Overall Length (at full-depth)	212"	217"
Maximum breakover angle	13°	13°
Number of cutting teeth	90	110

D.S.H.A. COMPLIANCE

Exceeds protective structure requirements of Paragraph 1906.1602 Federal Register Vol. 17, No. 66.

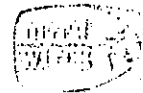


IMPORTANT:

Specifications are subject to change without notice. In some instances, exact dimensions and weights have been rounded. Equipment should be measured and weighed if exactness is required.

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The Professionals

Ditch Witch puts professionalism to work on your job . . . with trenching equipment built by professionals—and sold and serviced by professionals too. To the Ditch Witch dealer, trenching is a specialty—not a sideline. He is available with authorized parts for service and maintenance in the shop or field, on a 24-hour basis. Let The Professionals go to work for you!



A DIV. OF CHARLES MACHINE WORKS, INC. • P.O. BOX 66 • FERRY, OKLAHOMA 73077 • AG 0578-10-1

August 28, 1978

H.2.77

MEMO TO: James H. Havens, Director
Division of Research

FROM: H. F. Southgate
Chief Research Engineer



SUBJECT: Test Results of Daniel Boone
Parkway on 8-23-78

The test pit was located in the outside W.B. lane at Milepost 42 plus 896 feet. Rut depths prior to trenching was measured with stringline and measuring tape and found to be approximately 1.75 inches in both wheel tracks.

Road Rater tests were performed at the test pit, but the usual method was not employed. The tests were made with the sensors placed in a line perpendicular to the centerline. Analyses of these tests were somewhat confusing and will require further analyses.

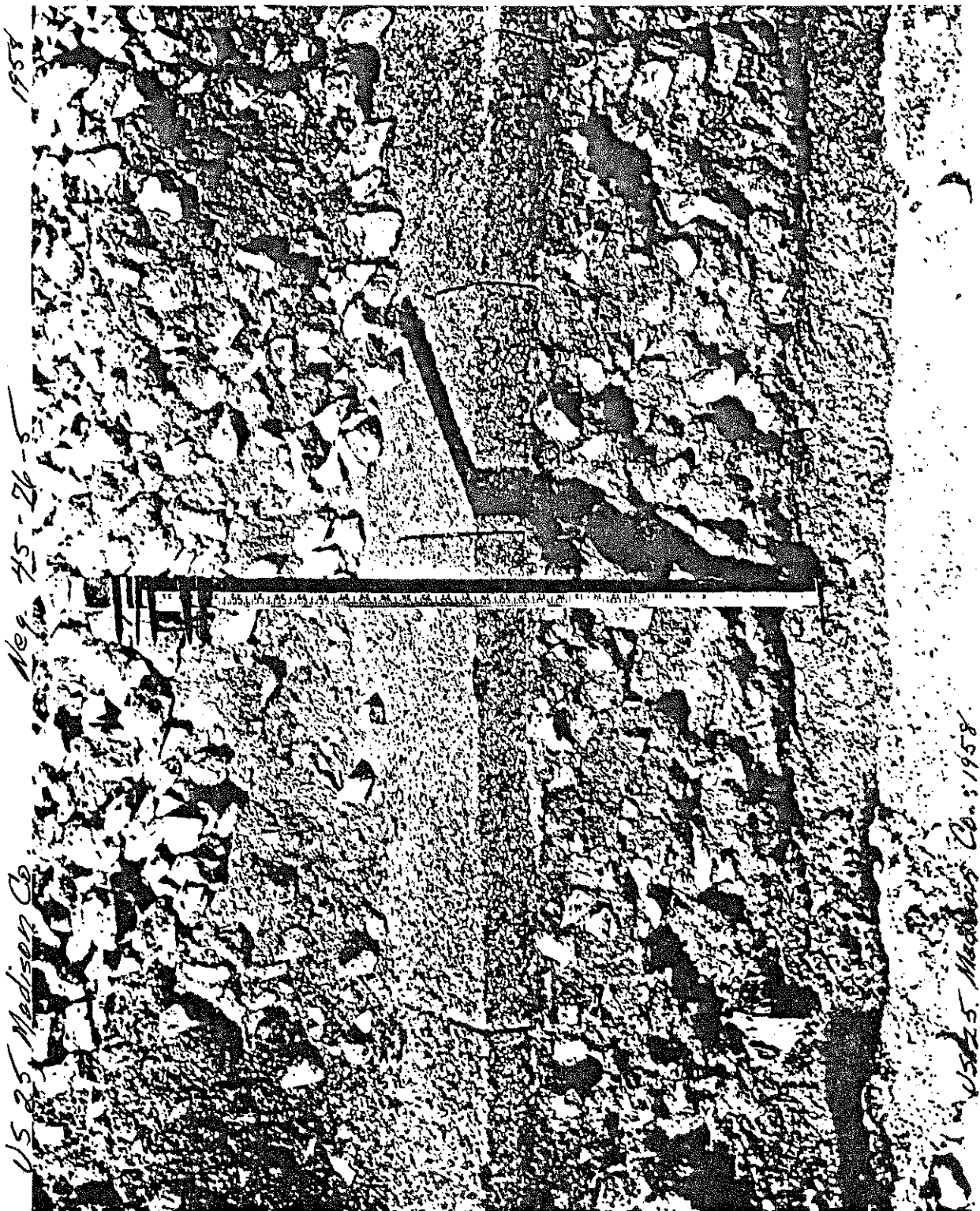
Road Rater tests were made at the 400-, 500-, 600-, 700-, and 800-foot marks from Milepost 42. The usual manner of testing was employed. At each point tests were performed in each wheel track, inbetween the wheel tracks, and at the edge next to the shoulder. Usually, thicker pavements will exhibit lower deflections. At each test location, the thinner pavement was in the wheel track area and exhibited noticeably lower deflections than the thicker area between wheel tracks and at the shoulder. Analyses indicates that the effective AC modulus in the wheel tracks is approximately 1.5 times the effective modulus of the thicker portions.

Inspection of the cut faces of the AC and subgrade indicated that there was no rutting of the subgrade under the wheel tracks. Indeed, construction planes could be found in the AC and tape measurements indicated virtually no change in the thickness of the bottom two lifts regardless of the location across the lane. However, tape measurements of the third lift showed some slight change in thickness -- thinner in the wheel tracks than other areas. The change in thickness of the fourth lift of the AC base was most pronounced and the differential was almost the same as the rut depth measured on the surface. The surface course exhibited almost no change in thickness anywhere across the lane.

Memo to: Jas. H. Havens, Director
August 28, 1978
Page two

In conclusion, my opinion is that 1) there was no rutting in the subgrade, 2) the rutting is a function of the steep grade, the slow moving very heavy loads which allowed sufficient time for progressive creep to occur, 3) the rutting was predominantly in the top lift of the base mix which did not have a sufficiently high enough stability to withstand creep actions, 5) I do not consider the rutting to be caused by instability of the surface course, 6) flow of the material from the wheel track areas to either side was a gradual and incipient particle-by-particle movement without reorientation of the aggregates. This flow was not the lava type of flow observed in the Tar section on KY 15 several years ago; and 7) analyses of Road Rater measurements indicate that the AC material was noticeably stronger in the wheel track areas than to either side. If so desired, graphs can be made to substantiate the analyses of the Road Rater test data.

kb



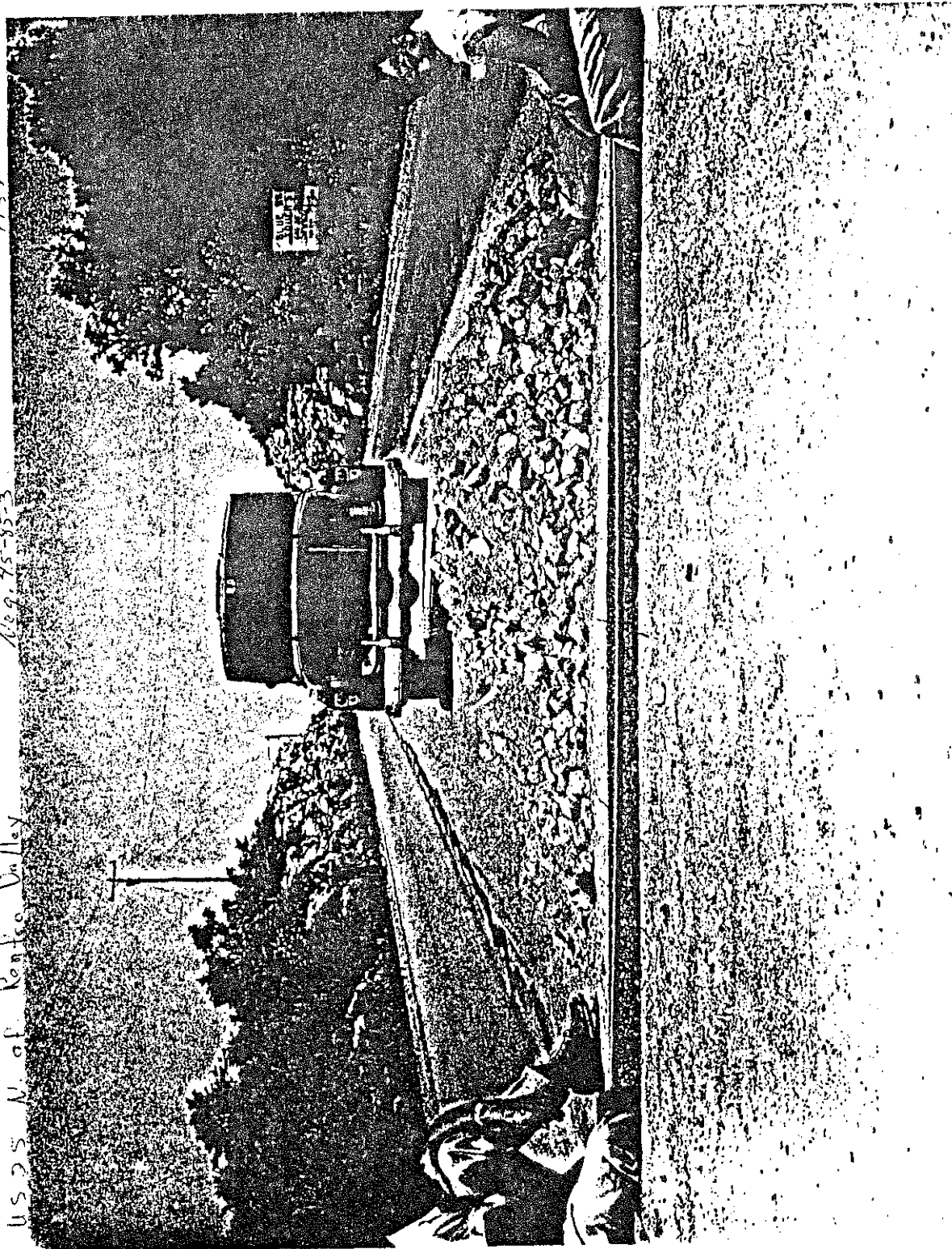
1958

Neg. 45-26-5

US 25 Madison Co.

US 25 Madison Co. 1958

US 25 N. of Ronto Valley Neg. 75-85-3 1958



US 25, N. of Ronto Valley; 1958

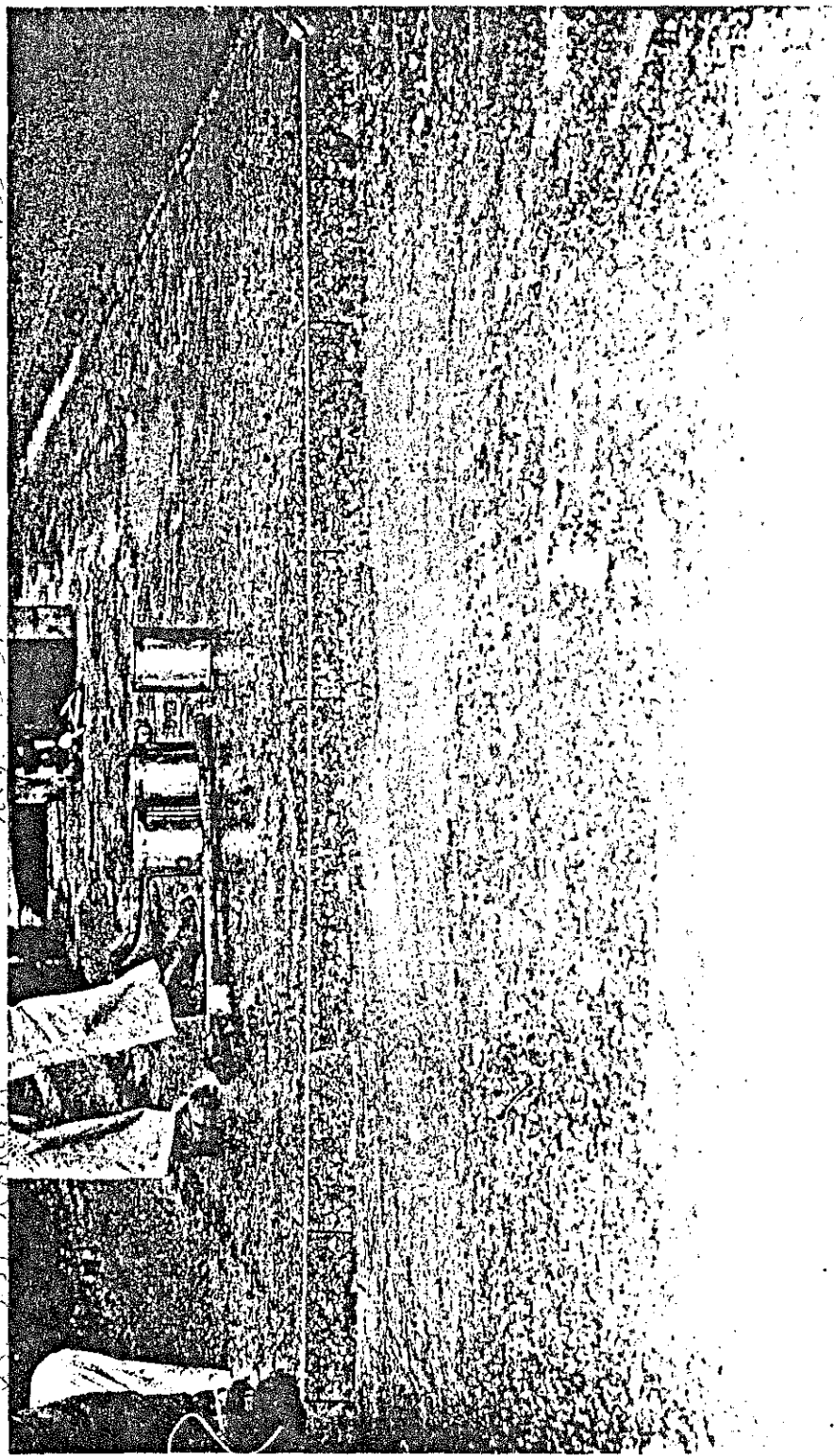
Drake & Havens, Fig. 18

45-76-4



US 25, Madison Co, 1958

US 25, London, 1958
Aug. 45-59-1

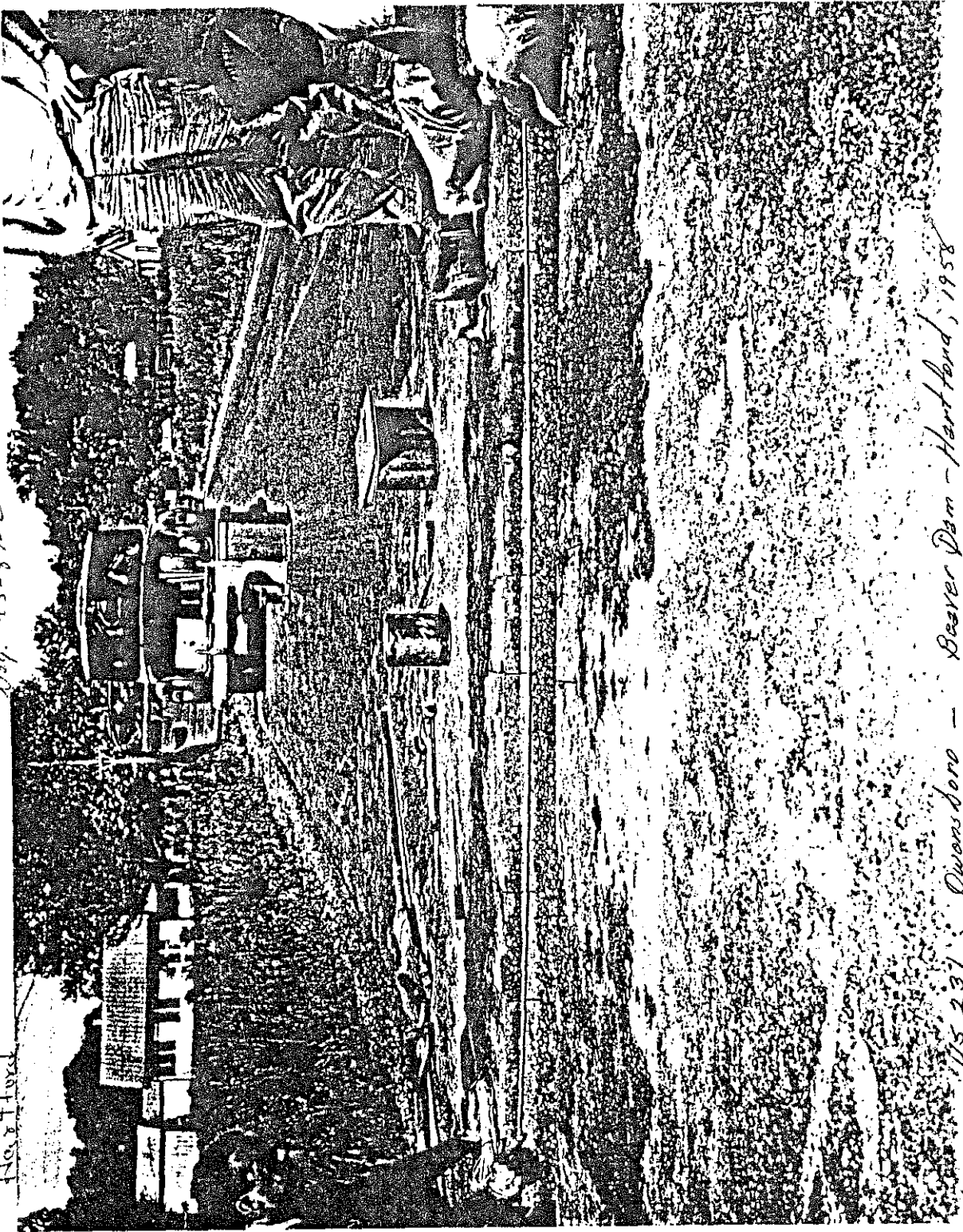


US 25, London, 1958

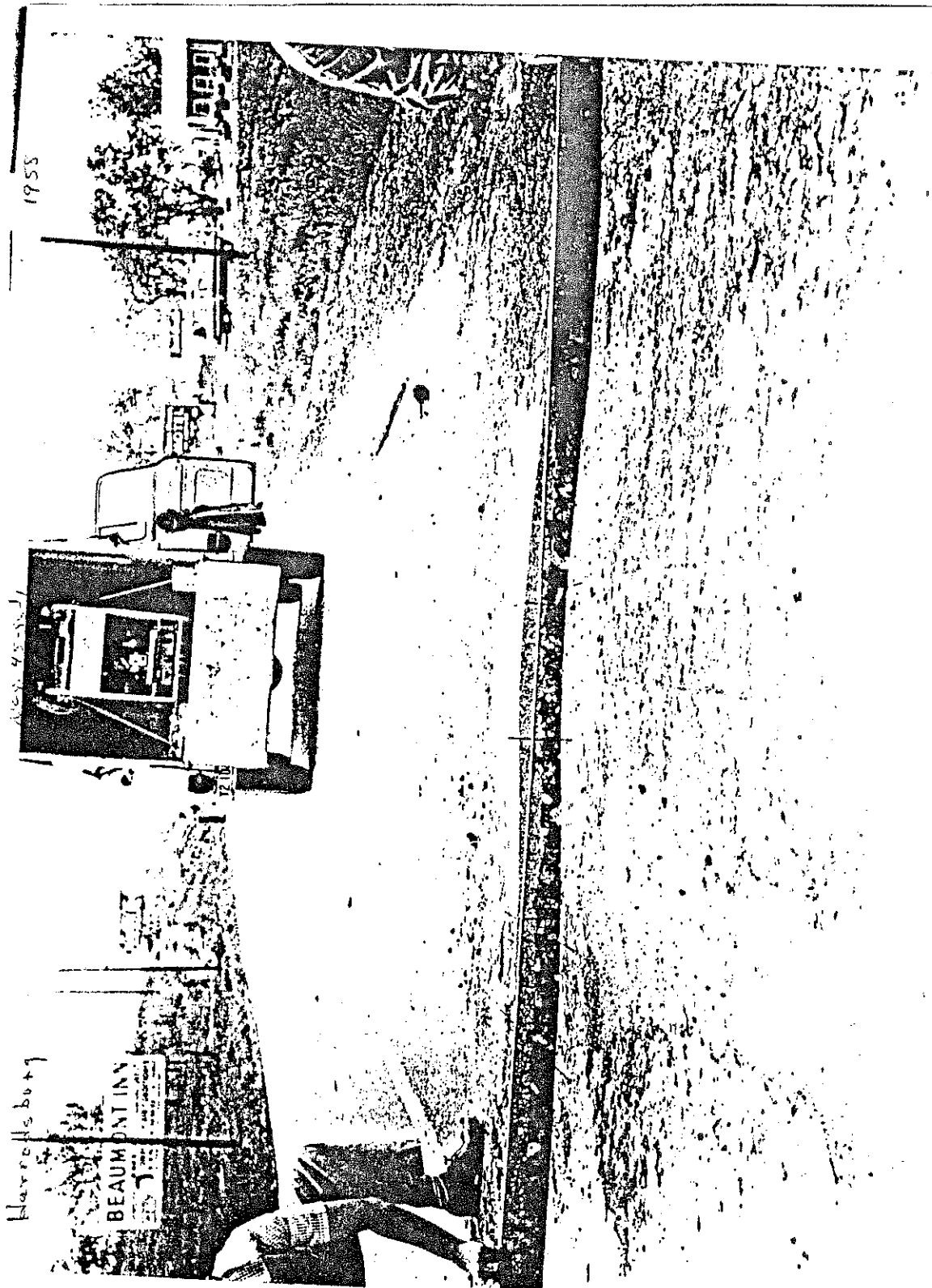
Hartford

Aug. 45-87-2

1958



US 237 1/2 Owensboro - Beaver Dam - Hartford, 1958

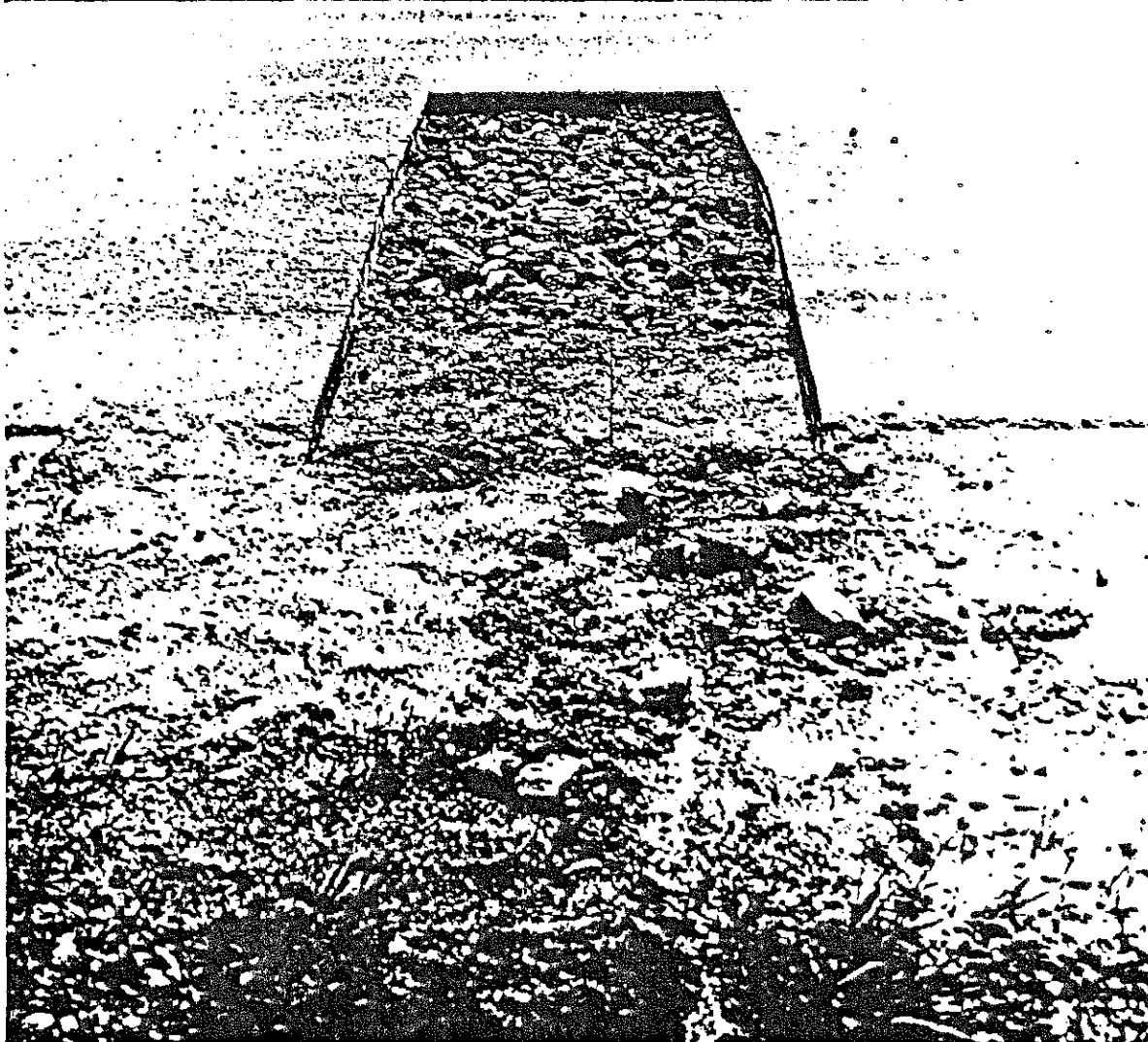


Harrisburg, 1958

US 25, Madison Co.

Neg. 45-16-2

1957

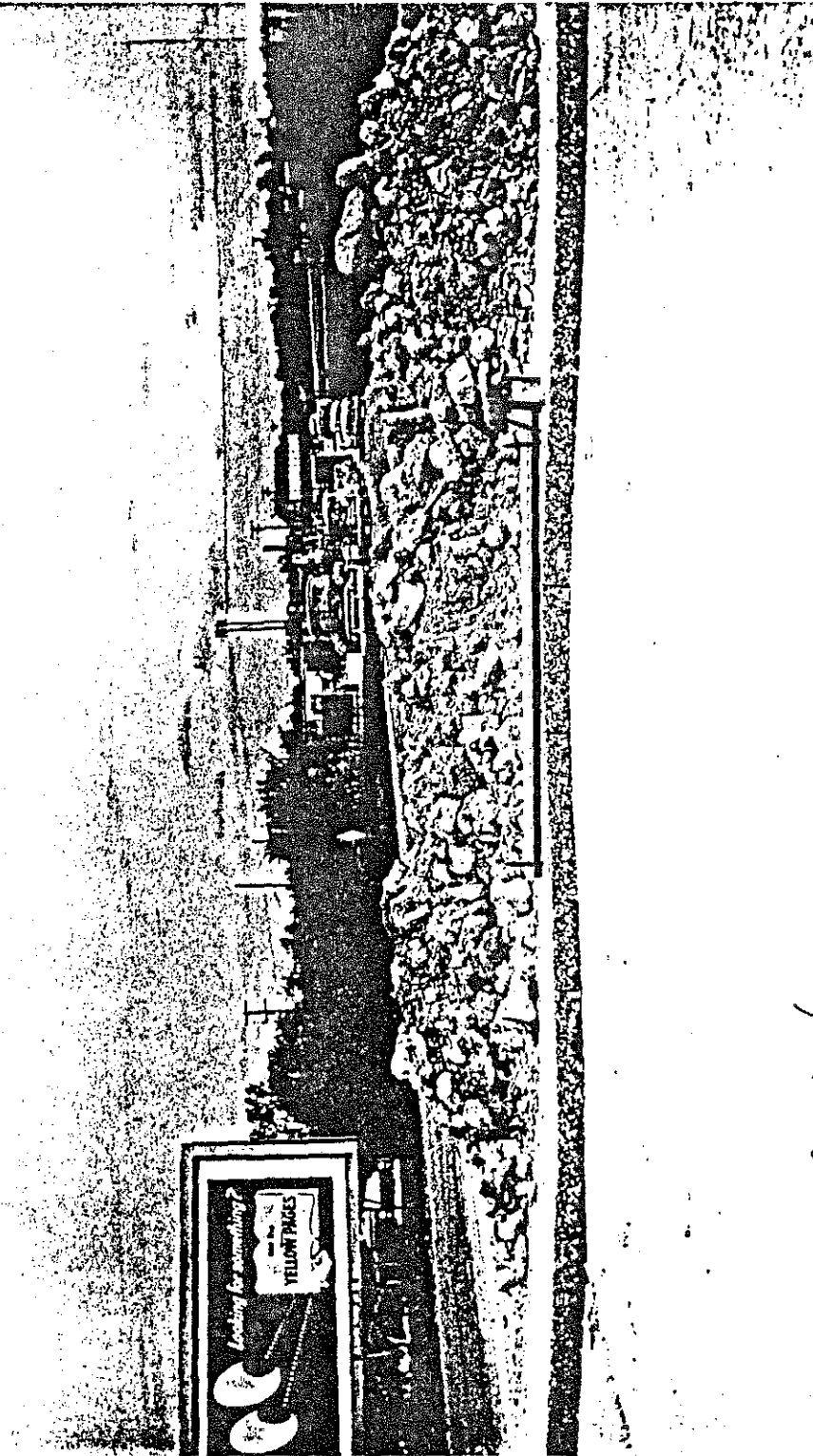


US 25 Madison Co, 1958

Ky. 4, N.E. of Lexington

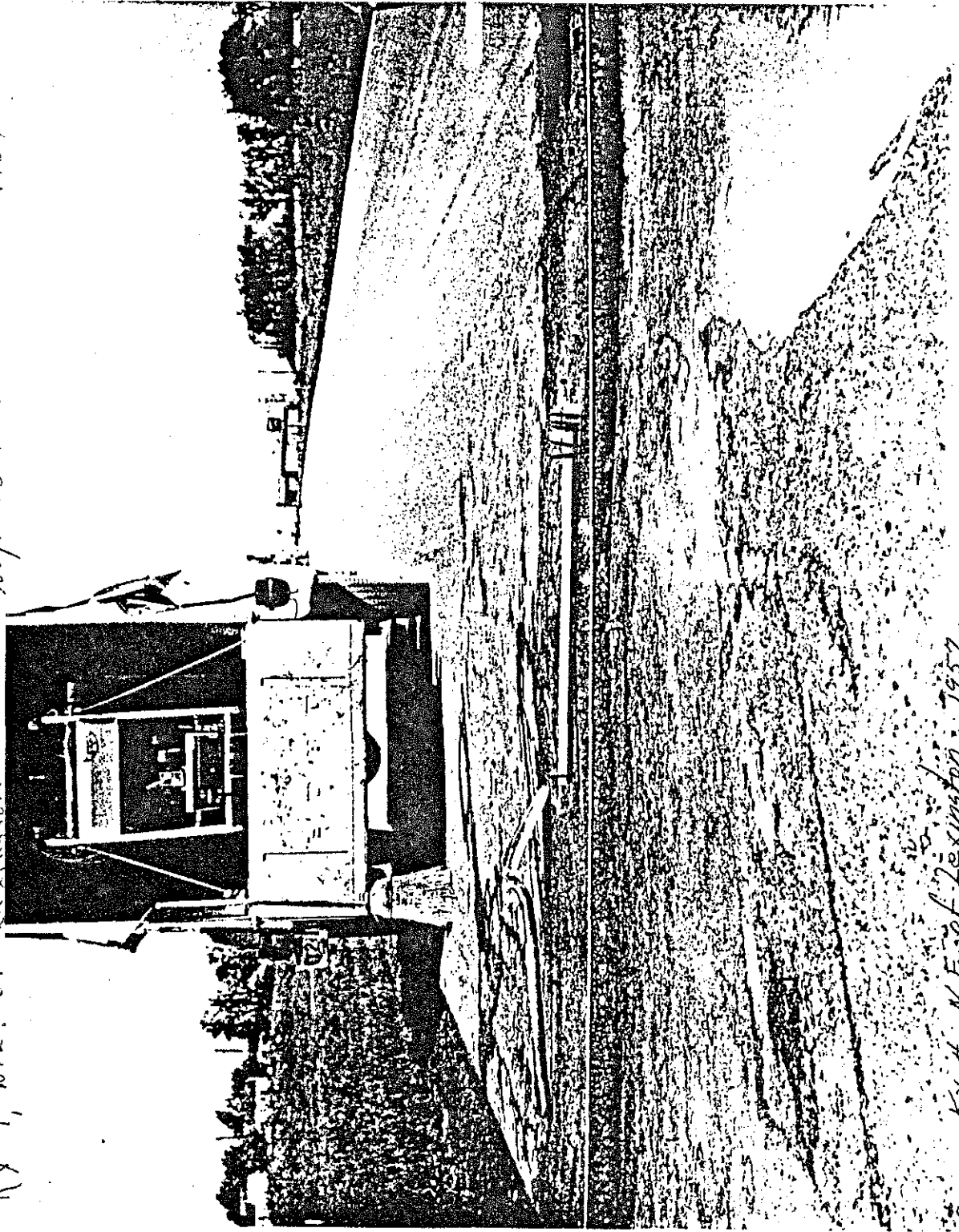
Neg. 45-77-3

1957



Ky 4, N.E. of Lexington, 1957

Ky 4, N.E. of Lexington
Aug. 45-77-1 1957



Ky. 4, N.E. of Lexington, 1957.

APPENDIX C

MEMO REPORT TO J. H. HAVENS, DIRECTOR OF RESEARCH;
FROM D. C. NEWBERRY, JR., CHIEF RESEARCH ENGINEER; DECEMBER 8, 1978,
Re: RUTTING INVESTIGATIONS; I 64 AND US 60
(FILES P-3-1 AND H-2-77).

December 8, 1978

P.3.1
H.2.77

MEMO TO: Jas. H. Havens, Director
Division of Research

FROM: D. C. Newberry, Jr. *DCC*
Chief Research Engineer

SUBJECT: Rutting Investigations; I 64 and US 60.

In regard to our further concern about rutting in the wheelpaths on asphaltic concrete pavements, two additional sites were trenched (cross-sectioned) and analyzed. One site was at MP 186.227, eastward on I 64, Boyd County. The rutting there had progressed to 0.75 inches or more. The second site was on the experimental, full-depth (18 inches) asphaltic concrete near Ashland (US 60) M.P. 8.139, at Summit. The rutting, as had been observed before, near Thousand Sticks on the Daniel Boone Parkway (also cross-sectioned)* appeared to have occurred in the form of shear in the upper five or six inches of the asphaltic concrete. The discovery of this manner of occurrence on the Parkway and the confirmation now of its typical pattern will have significant bearing on decisions and strategies employed in the design of pavement structures to carry heavy traffic. Labeled photographs are attached; and more detailed information follows.

Deflection tests were made with the Road Rater; density tests were made with the Seaman Nuclear Density Meter; and physical measurements were made from a string line at the surface.

The rutting was determined previously during a visit to the sites.** Rutting at the I 64 site was a maximum of 0.50 inches (12.7 mm) in the outer and 0.625 inches (15.88 mm) in

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Memo
December 8, 1978

the inner wheel track. Rutting at the US 60 site was a maximum of 1.19 inches (30.16 mm) in the outer and 1.125 inches (28.58 mm) in the inner wheel track. Both of these sites have a high volume of coal-truck-type traffic.

Photograph 1821-6 shows obvious cracking in the inner lane. This cracking resembles that examined on I 64 in Clark and Montgomery Counties in 1968.*** That portion of I 64 was overlaid in 1973. The cracking here and there extends only through the surface. The cracking now, as then, is believed to have been induced by rolling -- at the time of construction. Cracking is less obvious in the outer lane -- more especially in the wheel paths. There, those cracks (see Photo 1821-5) appear to have been healed by traffic; and close-spaced, tension cracks perpendicular to the wheel path indicate tractive displacement (shear) in the backward direction. This type of movement was observed at the Daniel Boone Parkway site and has been observed on I 75 at about MP 51 and northward. Lines were scribed onto the surface at the I 64 site (see Photo 1821-14); they will be observed through the next warm season.

The results of the density and physical cross-section measurements are graphically displayed, and the graphs are attached hereto.

The Road Rater data are available but are not included here.

The data and photographs support the following observations: 1) the rutting is contained in the upper asphaltic concrete courses; 2) the I 64 cross section, measuring the depth of the various courses from a string line, indicated possible but slight rutting in the DGA base course.

Additional attention must be given to achievement of higher stabilities in the upper pavement courses to assure immunity against rutting. AC 20 or heavier asphalt cement should be used in the upper portion of the heavy duty pavements. In fact, the use of AC 40 may be indicated.

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December 8, 1978

The trenching of I 64, using the earth saw which cuts dry, upon exposure of the DGA base layers revealed no indication of free water or muddiness anywhere. Photograph 931-7, of the I 64 cut, is a good view of the DGA layer.

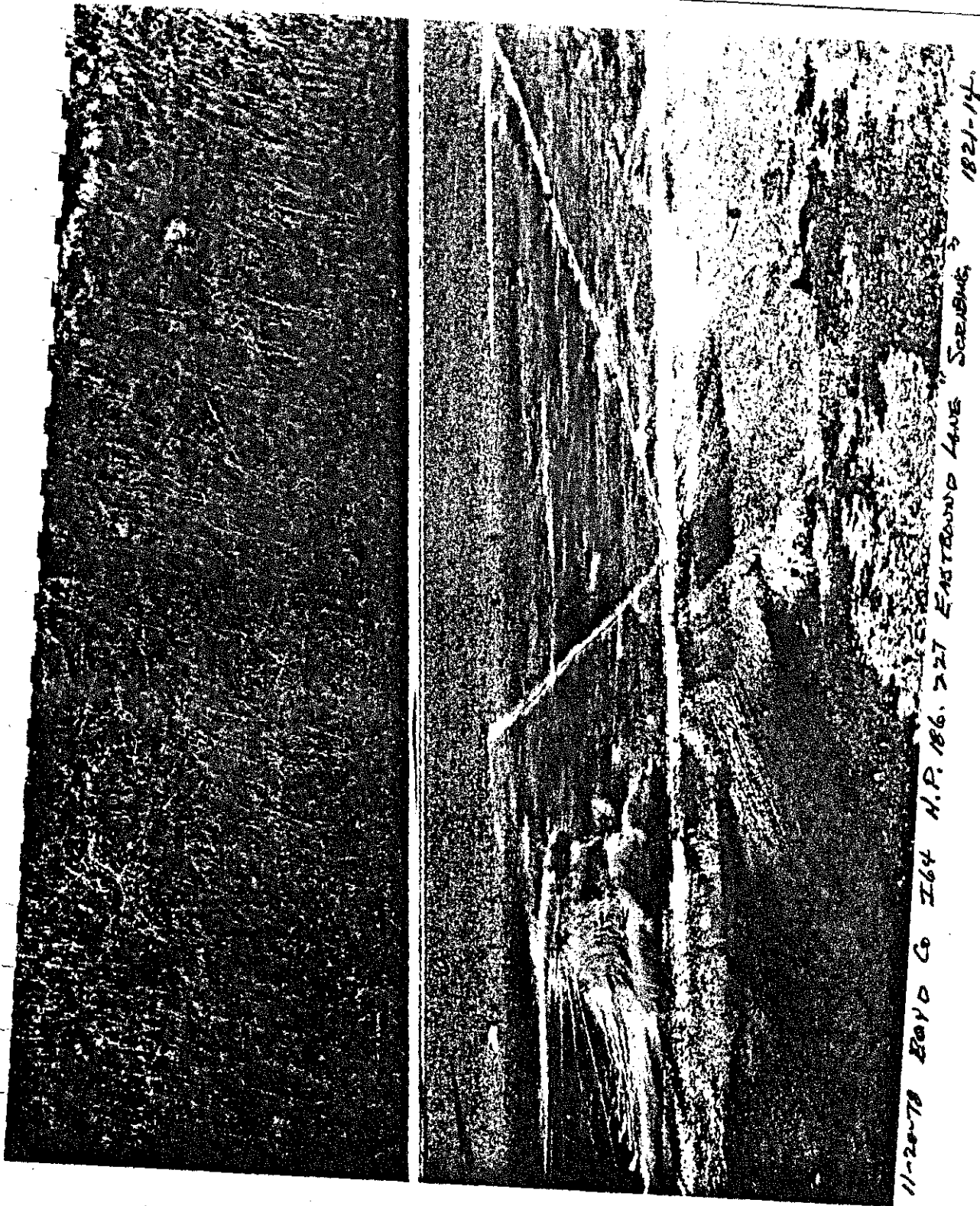
Photograph 931-2, of the US 60 cut, exposes the 18-inch, (457 mm) asphalt concrete depth and the surface rutting there. No free-draining water was found in or around the full-depth section.

gd

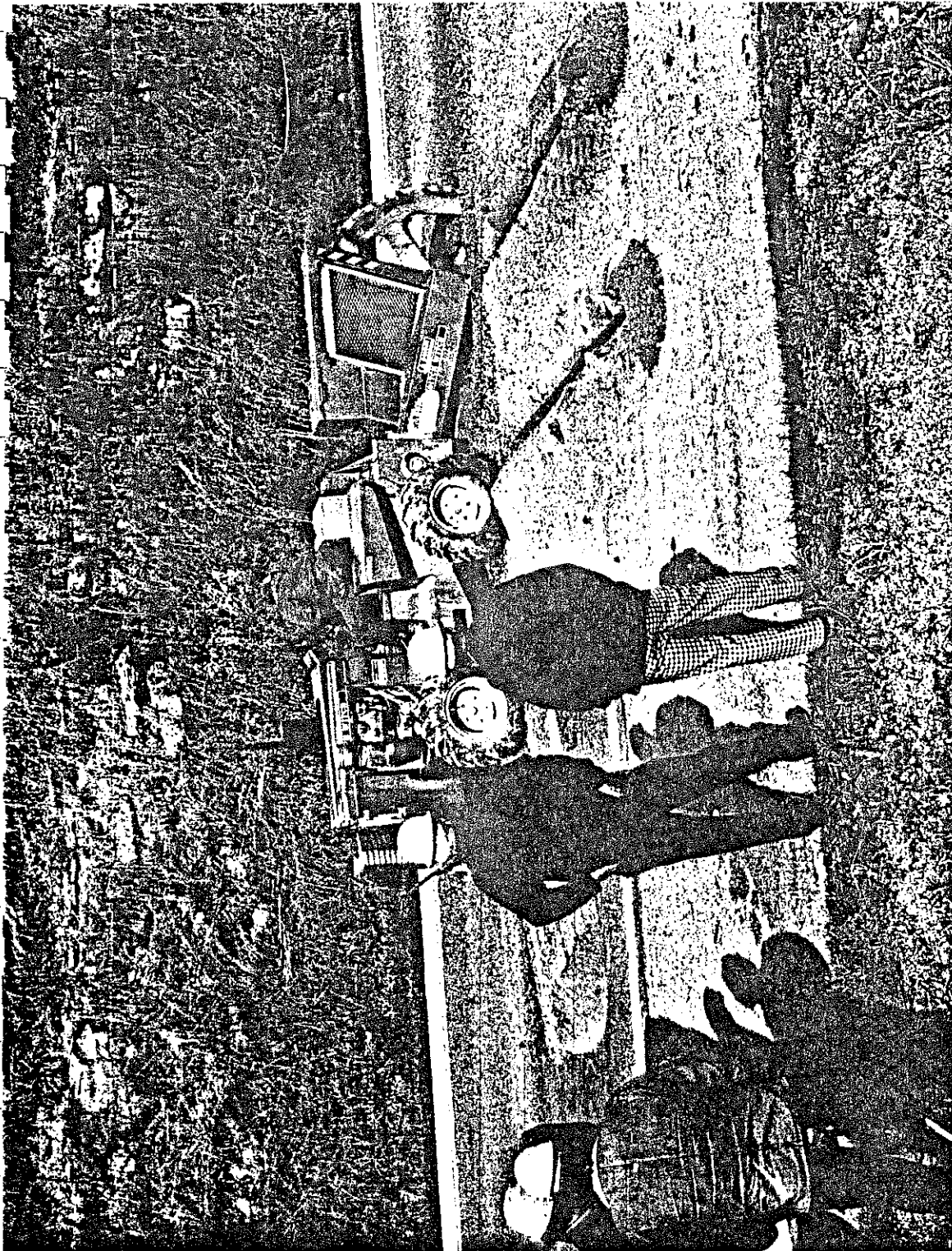
*Memorandum, September 5, 1978; File P.3.1; J. H. Havens to W. B. Drake; Subject: "Rutting, Asphaltic Concrete Pavements," with attachments.

**Memo to G. F. Kemper from A. R. Romine, August 24, 1978; Inspection of I 64; Rowan, Carter, and Boyd Counties.

***Memo report by D. C. Newberry; August 20, 1968; also: Unfinished Report; "An Investigation of Surface Cracking in a Bituminous Concrete Surface [I 64-5-(8)100];" Jas. H. Havens; February 1970; and photos made 3-9-72.



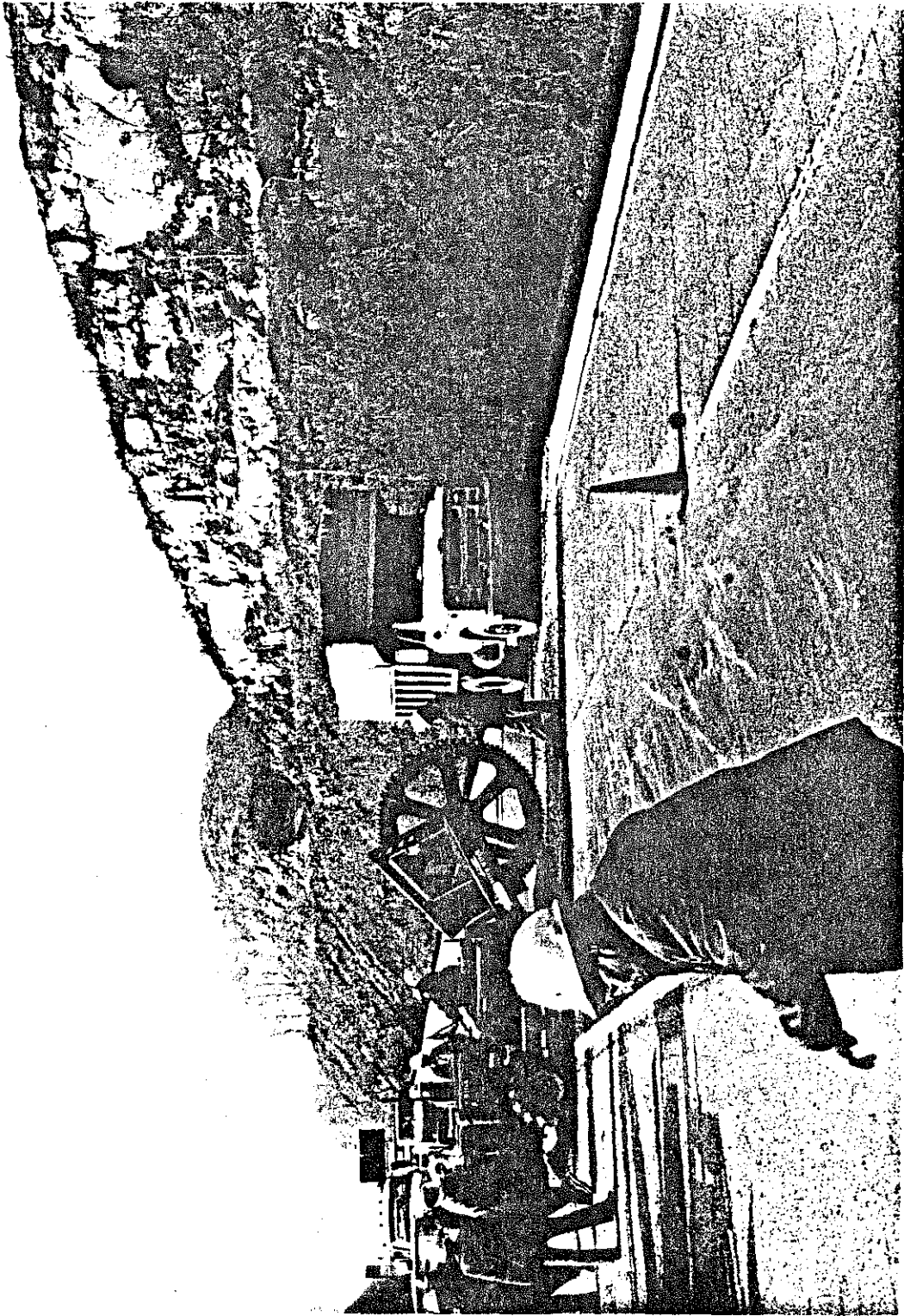
11-20-78 BOYD Co I64 N.P. 186. 227 EASTBOUND LANE SCENIC 1821-14.



11-20-78 Boyd Co. I 64 HP 186.227 EMBROID LANE



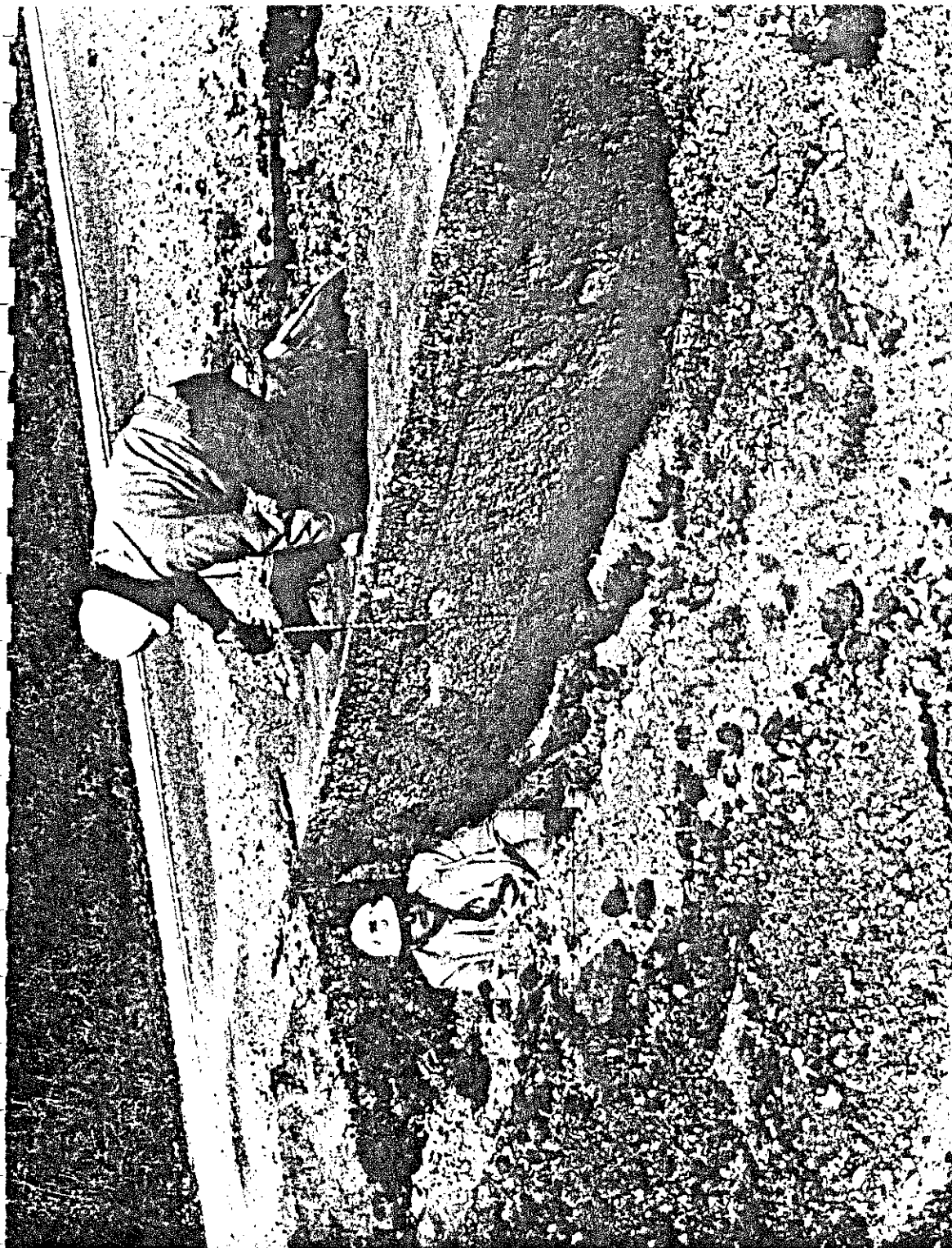
11-20-78 GORDON HP 186.227 EASTBOUND LANES I-64



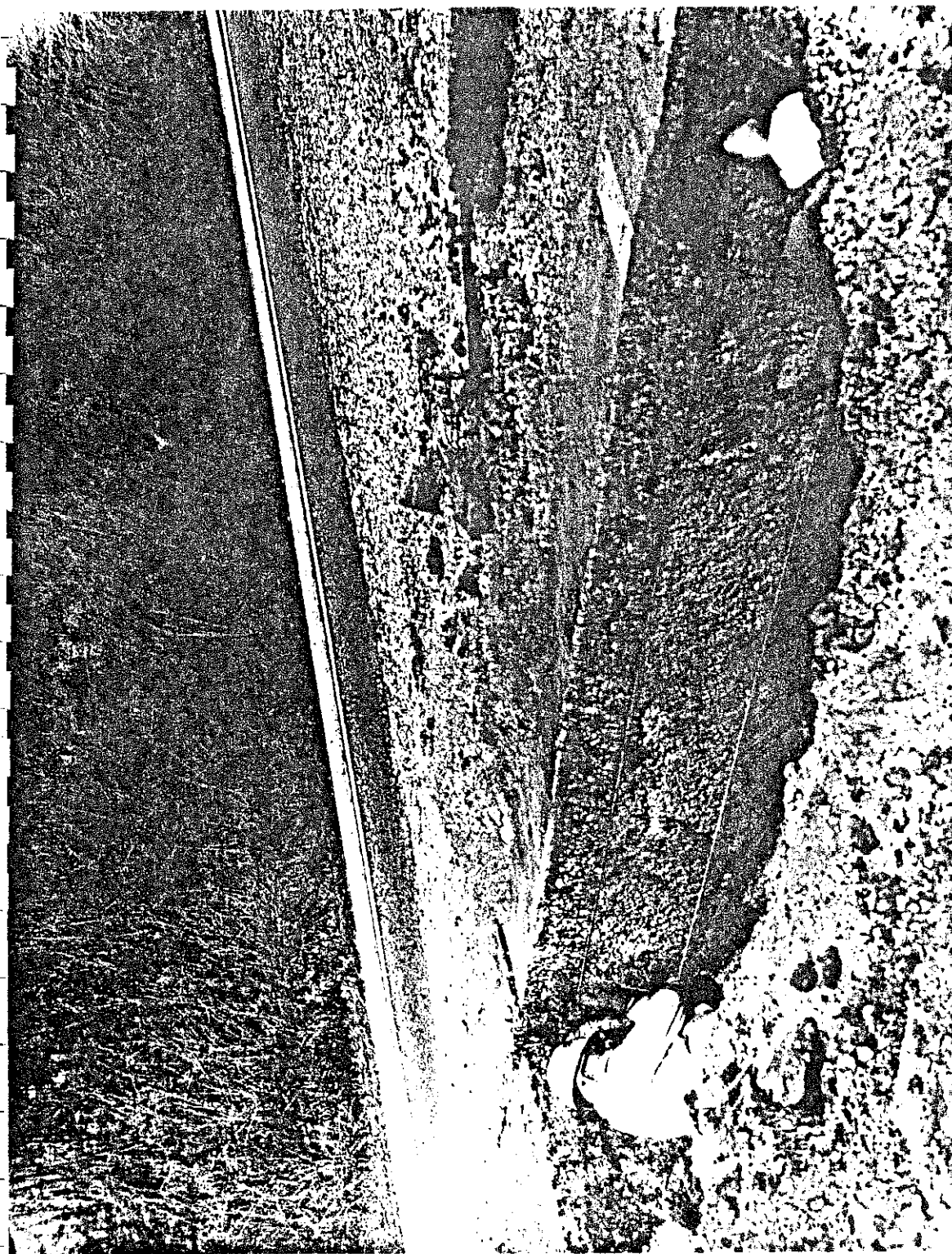
11-20-78 13000 00-4P 186.227 EASTBOUND 164



11-20-78 B&O Co. I 64 HC 186. 227 EAST BOUND LANE



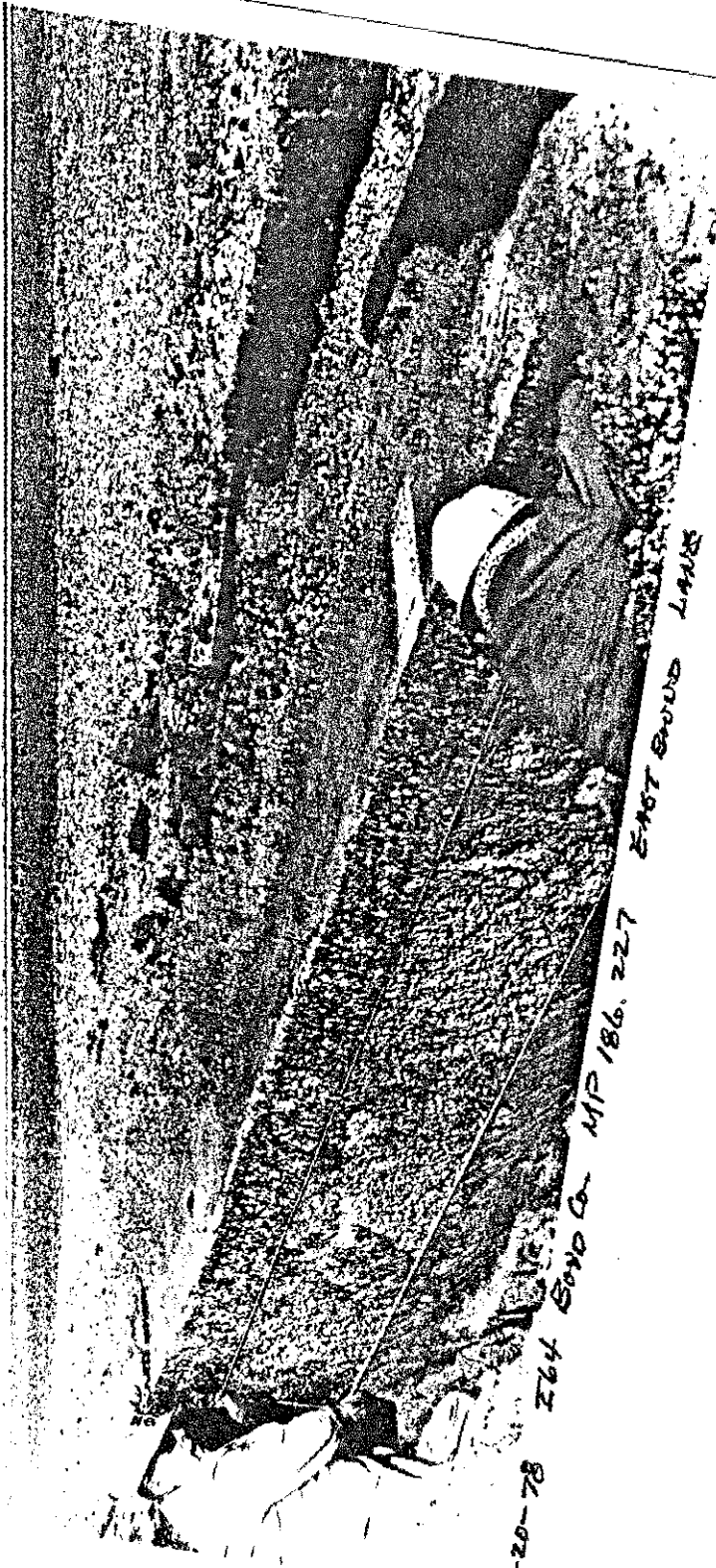
11-20-78 GND Co I64 MP 186.227 EASTBOUND LANE



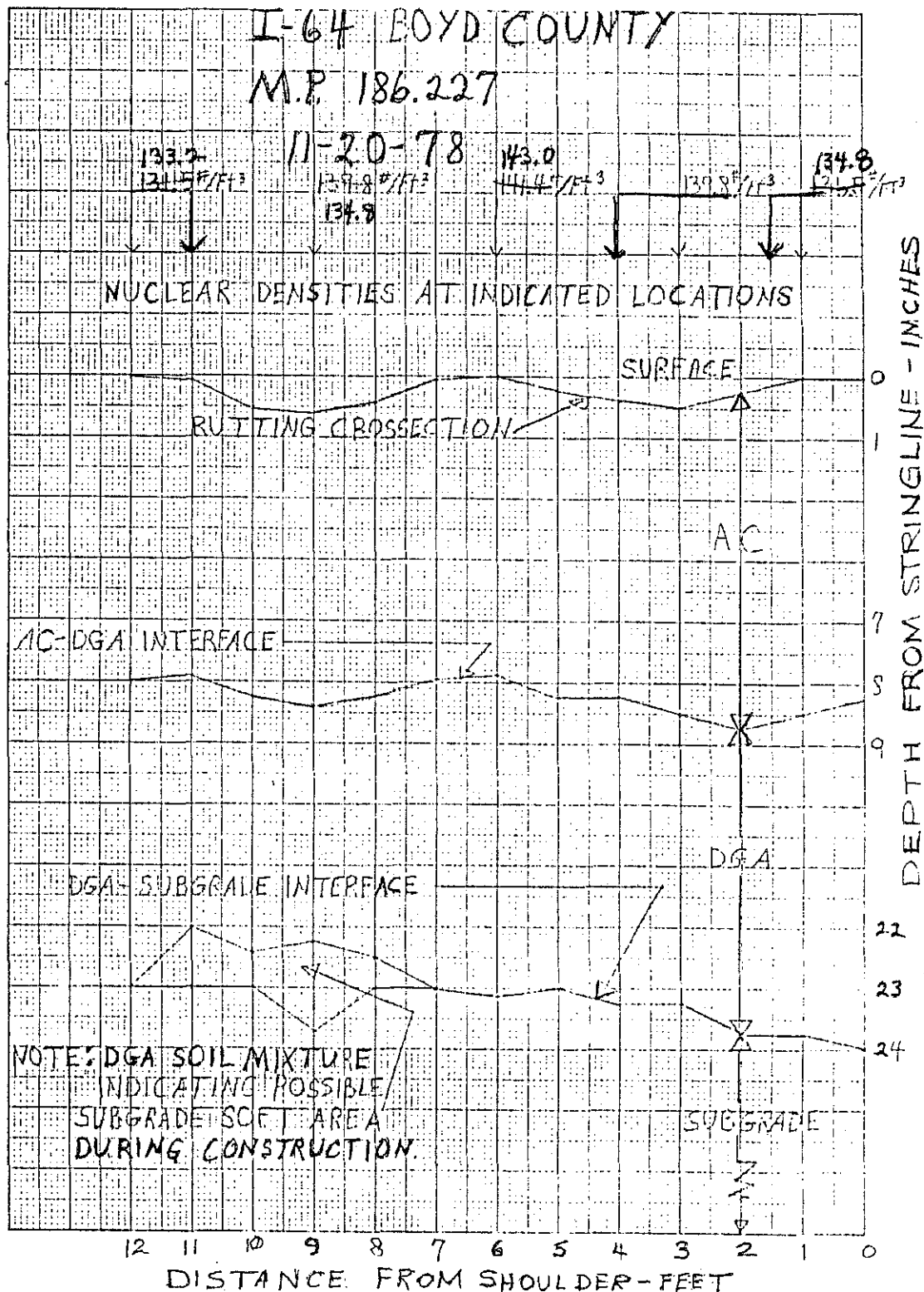
11-20-78 BRAD CO T64 MP 186.227 EASTBOUND LWS

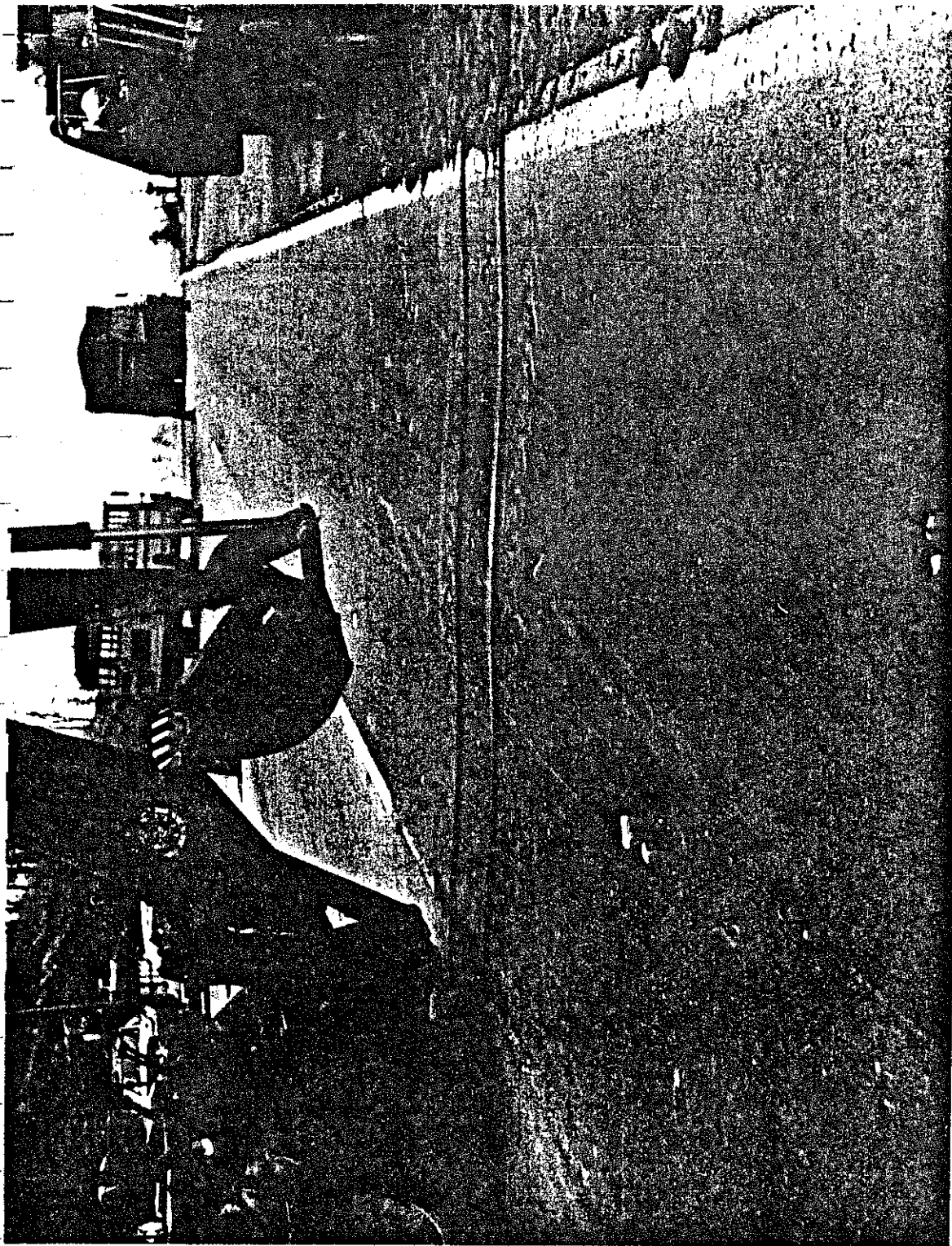


11-20-78 Boy's Co. I 64 HP 186.227 East 80000 Lane

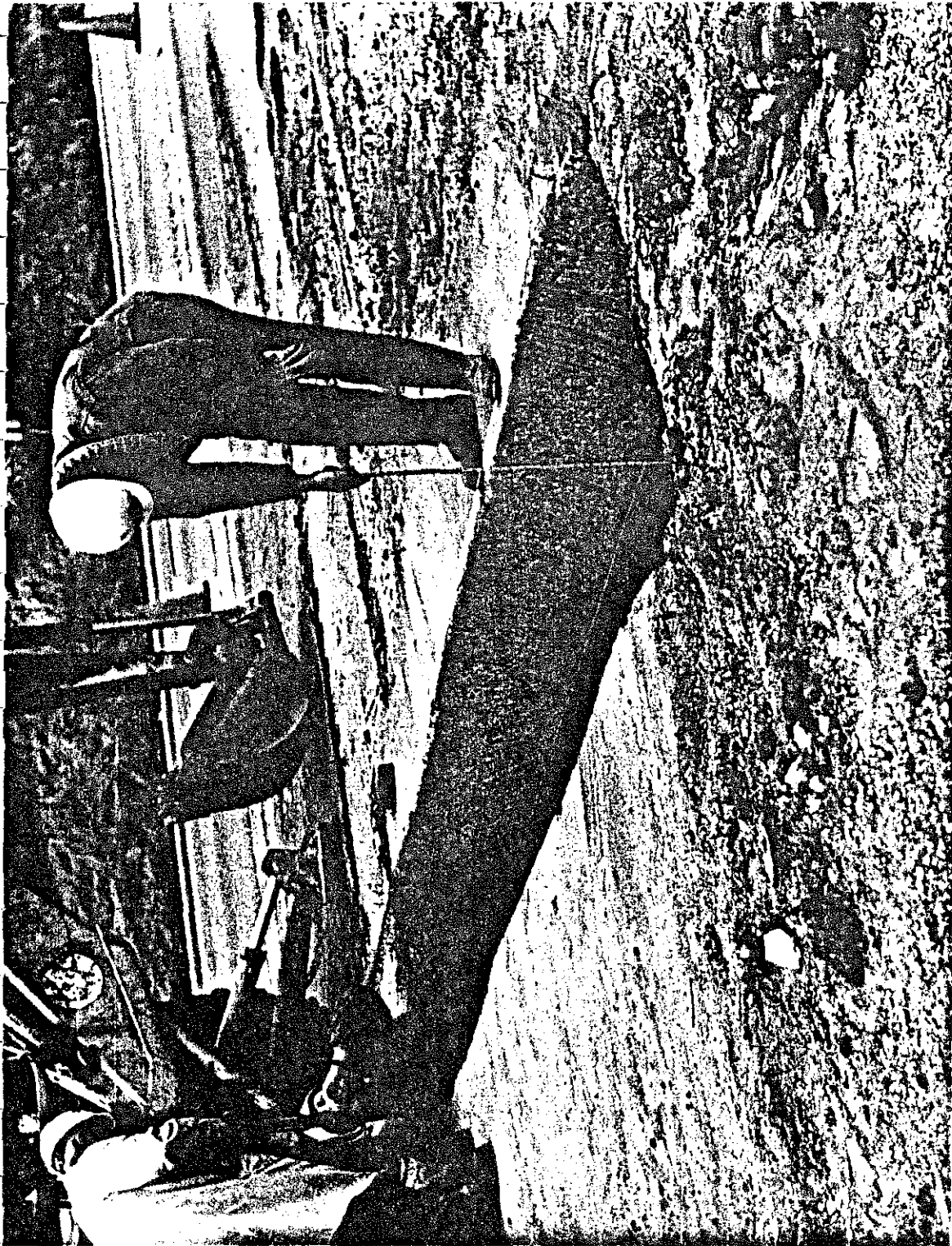


11-20-78 264 GONDON MP 186.227 ENOT GOOD LAND

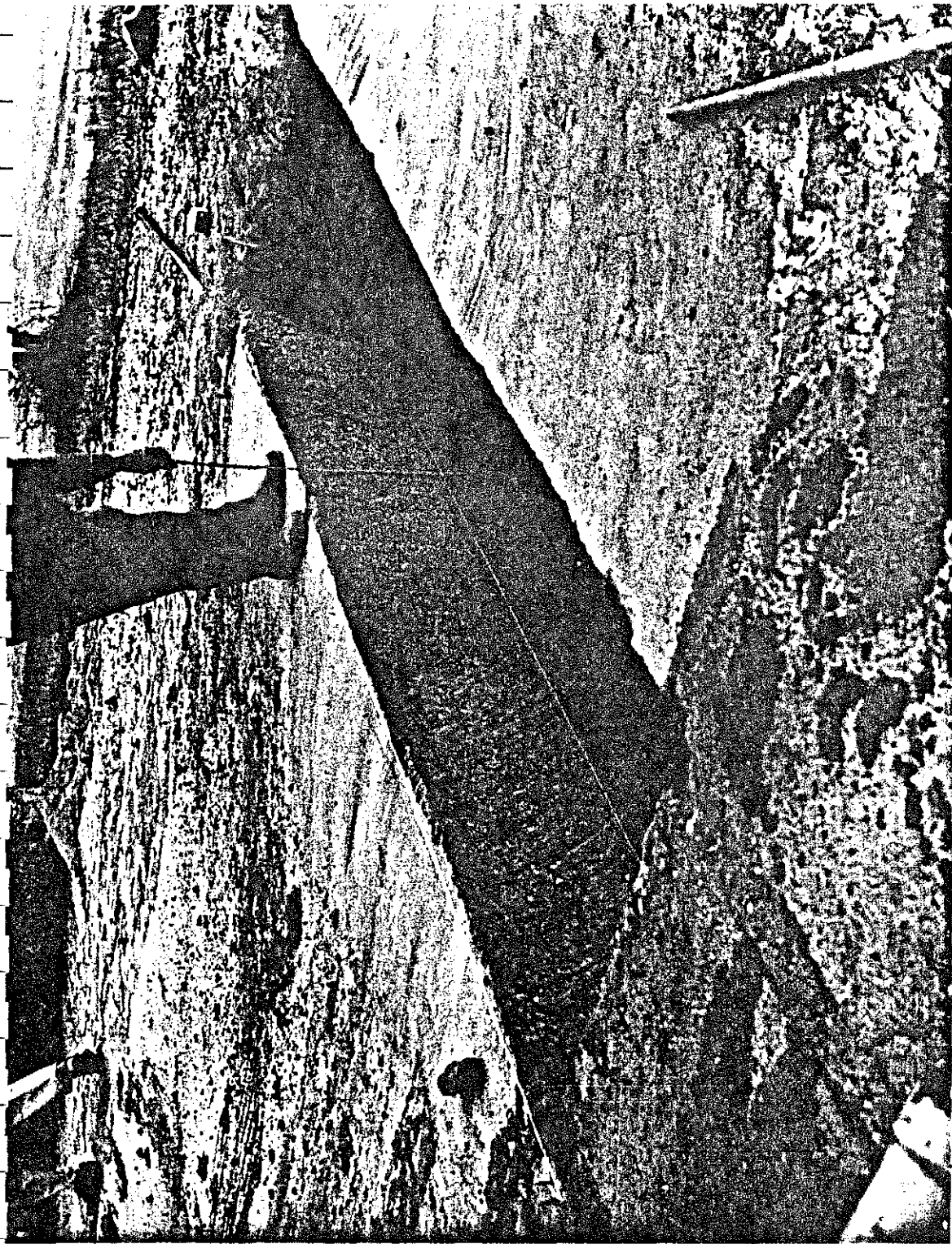




11/21/78 Bay Co. US60 MP 8.139 EASTBOUND LANE REMOVING SLAB.



11-21-78 Boyd Co. 0560 HP 8.139 East Bound Lane 18" Full Depth



11/21/78 B-40 G. US60 HP B. 139 EASTWARD LAKE 18" FULL DEPTH

